Effects of Cone Tip Roughness, *In-Situ* Stress Anisotropy and Strength Inhomogeneity on CPT Data Interpretation in Layered Marine Clays: Numerical Study

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Abstract: This paper explores the effects of cone tip roughness factor, soil *in situ* stress ratio and strength non-homogeneity on cone tip resistance in three-layer clay deposits, aiming at presenting a general CPT data interpretation framework to extract soil layered profiles and strengths. Large deformation finite element (LDFE) analysis has been carried out, simulating continuous penetration of the standard cone penetrometer from the seabed surface, with the consideration of cone tip roughness factor, *in situ* stress ratio and soil strength non-homogeneity within a practical range. In the LDFE analyses, a thin relatively stiff or soft clay layer was interbedded in a uniform or NC clay deposit, with various strength ratios between two successive layers and thickness ratios of the thin layer to the cone diameter explored. The results have shown that the noted factors have significant influence on net cone penetration resistance in stratified deposits. These influences in the layered deposits were found to be

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similar in magnitude to those on the cone limiting resistance in a single clay layer, leading to the resultant influence on a thin layer correction factor for cone penetration resistance minimal. This was confirmed by the evidence of the evolving soil flow mechanisms around the cone tip. Therefore, the design charts and formulas developed for a smooth cone in uniform three-layer soils are applicable for delineating layer boundaries and interpreting the undrained shear strength of each identified layer for a partially rough cone in three-layer non-homogeneous clay deposits. However, the cone tip roughness factor was shown to have an influence in the layer boundary identification, especially for the cone penetration from a stiff layer to a soft layer.

Keywords: Cone penetration tests; Marine clay; Layered soils; Numerical analysis; Penetration resistance; Site investigation

27 Introduction

28 **Background**

29 The relentless rise in world energy consumption has led to hydrocarbon exploration in coastal
30 and offshore areas in unexplored and undeveloped regions. The structures required for the
31 offshore exploration need strong foundations. Understanding the soil characteristics and their
32 geological settings on the seabed is crucial to offshore foundation design. Cone penetration test
33 (CPT) is a common site investigation tool to provide continuous soil strength profiles without
34 soil sample coring and laboratory tests.

35 Offshore soils are exhibiting complex soil conditions due to the geological history of the
36 seabed. In emerging provinces and fields, highly layered deposits are prevalent, where accurate
37 identification of layer boundaries and strength of each identified layer are critical for economic
38 and safer designs of foundations and anchoring systems of offshore structures. For site
39 investigations, the need to rely more on *in situ* penetrometer tests, compared to laboratory tests
40 on cored samples, has been emphasized due to near continuous profile and absence of soil
41 sample disturbance issue (Karakouzian et al. 2003; Cai et al. 2011). Among in situ tests, the
42 piezocone penetration test (CPTu) is the most commonly used one, giving continuous profiles
43 of (a) cone tip resistance, q_c ; (b) sleeve friction, f_s ; and (c) pore water pressure, u_2 . Cone tip
44 resistance is of particular interest of this paper.

45 The standard field cone penetrometer is cylindrical in shape with a conical tip that has a base
46 area of 10 cm^2 (diameter $D = 35.7 \text{ mm}$) and a 60° tip-apex angle. The penetration rate of $v =$
47 20 mm/s ensures undrained conditions in clay deposits. As such, the intact undrained shear
48 strength s_u is derived from the net cone tip resistance q_{net} as

$$49 \quad s_u = \frac{q_{\text{net}}}{N_{\text{kt}}} = \frac{q_t - \sigma_{v0}}{N_{\text{kt}}} \quad (1)$$

where N_{kt} is the cone bearing capacity factor, q_t is the total cone tip resistance and σ_{v0} is the total overburden stress at the level of the cone shoulder, d (see Figure 1). For this study, the cone was simulated with a solid smooth shaft and hence $q_t =$ computed tip resistance q_c .

Previous Works

A large number of investigations have been carried out on cone penetration response in single layer clay with uniform undrained shear strength (Teh and Houlsby 1991; Lu et al. 2004; Ma et al. 2014). The cone bearing factor, N_{kt} , was found to be a linear function of soil rigidity index, I_r , and cone tip roughness factor α , as shown below (Ma et al. 2014; 2015)

$$N_{kt} = 3.47 + 1.56 \ln I_r + 1.3\alpha \quad (2)$$

Note, N_{kt} values are independent of cone shaft roughness factor (Lu et al. 2004), and as such in this paper, roughness factor refers to cone tip roughness, while that of cone shaft is assumed as fully smooth ($\alpha = 0$).

The effect of natural soil layering on cone penetration response (and hence delineation of interfaces between soil layers and interpreting actual undrained shear strength) has attracted relatively less attention, and mostly limited to sediments with either surface or interbedded sand layer. Investigations through numerical modeling have been conducted by van den Berg (1994), Ahmadi and Robertson (2005), and Xu and Lehane (2008); and through analytical approaches by Vreugdenhil et al. (1994), and Mo et al. (2014, 2017).

For stratified clay deposits, recently, Walker and Yu (2010) carried out analysis for two-layer stiff-soft and three-layer uniform stiff-soft-stiff clays. Large deformation finite element (LDFE) analyses were carried out using the commercial FE package Abaqus/Explicit. However, no quantitative design expressions were given for practical use in characterizing the stratified sediments.

Where a thin soil layer is interbedded in a soil deposit, the cone penetration depth may not be sufficient to mobilize its full resistance in that layer. Thus a correction factor is needed to interpret actual strength of the thin layer. Following Robertson and Fear's (1995) and Moss et al.'s (2006) suggestions, the correction factor k is defined as the ratio of the actual full net resistance in the thin embedded layer $q_{\text{net},f}$, to the measured net resistance in that thin layer $q_{\text{net},m}$

$$k = \frac{q_{\text{net},f}}{q_{\text{net},m}} \quad (3)$$

Ma et al. (2015, 2016) describes a new approach for interpreting fully smooth cone penetrometer data in soft-stiff-soft clay deposits, based on an extensive parametric study using LDFE analyses. It was found that, regardless of strength ratio and rigidity index ratio between two successive layers, the interface of soft-over-stiff layers can be identified at $0.8D$ (D is the cone diameter) below the kink in penetration resistance curve in the top soft layer. The interface of stiff-over-soft layers can be demarked at $1.3D$ above the kink in penetration resistance profile in the bottom soft layer.

An extensive parametric study was also conducted later by Ma et al. (2017) using LDFE analyses for a smooth cone in stiff-soft-stiff clay deposits. The results were used to establish the interpretation framework. Regardless of the strength and rigidity index ratio between two successive layers, when the cone penetrates from stiff to soft clay layer and the reverse, the layer interface can be demarked respectively at $1.3D$ (D is the cone diameter) above and $0.8D$ below the gradient discontinuity on the penetration resistance profile in the soft layer. These are consistent with the conclusions obtained for cone penetration in soft-stiff-soft soils (Ma et al. 2015). The undrained shear strengths of the top and bottom (stiff) layers can be interpreted using a single layer approach and the resistance profile without the influence of the adjacent soft layer. The interpretation for the interbedded 2nd (soft) layer necessitates implementing a

correction factor, which was shown to be a function of the relative thickness of 1st and 2nd layers, and the rigidity index and strength ratio of the adjacent layers.

By combining the results of the standard smooth cone penetration in soft-stiff-soft clays and stiff-soft-stiff clays (Ma et al. 2015, 2017), a complete design chart is proposed for interpreting a thin clay layer (either soft or stiff) sandwiched in a thick clay layer, as shown in Equation 4 and Figure 2 (Ma et al. 2017).

$$k = \begin{cases} 1.08 - 0.21x + 0.019x^2 & \text{for } x \geq 0.35 \text{ for stiff-soft-stiff clays} \\ 0.91 + 0.31x - 0.016x^2 & \text{for } x \geq 0.35 \text{ for soft-stiff-soft clays} \\ 1 & \text{for } x < 0.35 \text{ for both} \end{cases} \quad (4)$$

$$x = \left(I_{r,\text{soft}} \right)^{0.2} \times \left(\frac{I_{r,\text{stiff}}}{I_{r,\text{soft}}} \right)^{0.3} \frac{\left(s_{u,\text{stiff}} / s_{u,\text{soft}} \right)^{0.5}}{(t_2 / D)}$$

Overall in all the previous studies for CPT in layered clay deposits, cone tip roughness factor $\alpha = 0$ (fully smooth) has been considered, with the three-layer soil profiles consisting of uniform clay in each individual layer. However, from the studies of cone penetration in single clay layer (Lu et al. 2004; Ma et al. 2014), cone tip roughness factor has shown to have significant influence on the cone bearing factor. Furthermore, from Lu et al. (2004), it is clear that the cone bearing factor is also affected by the *in situ* stress ratio, $\Delta = (\sigma_{v0} - \sigma_{h0})/2s_u$. In addition, undrained shear strength profiles of some soft, normally- and lightly over-consolidated clay layers increase more or less linearly with depth (i.e. gradient $k_s > 0$; Randolph and Gourvenec 2011). As such, their effects on cone penetration resistance need to be studied. The significance of this paper therefore lies in quantifying the influences of these factors, not considered by Ma et al. (2015, 2017), and proposing according adjustments to the CPT data interpretation approaches. This will turn the approaches to more robust ones and ensure wide applicability in practice for designing foundations and anchoring systems.

Objective of Present Study

This paper describes the results from LDFE analyses undertaken to provide insight into the effects of cone tip roughness factor α , *in situ* stress ratio Δ , and soil strength non-homogeneity factor $k_s D/s_{um}$ on CPT data interpretation in layered clays. Continuous penetration of the standard piezocone penetrometer from the seabed surface has been considered. In all the analyses, both soft-stiff-soft and stiff-soft-stiff deposits have been considered, covering a range of strength ratios between two successive layers and thickness ratios of the middle layer to the cone diameter.

Numerical Analysis

Geometry and Parameters

A cylindrical cone penetrometer penetrating into both soft-stiff-soft and stiff-soft-stiff three-layer clay deposits were analyzed. Figure 1a ~ Figure 1e illustrate the schematic diagram of the study plan, where the penetrometer has a diameter D and the undrained shear strength and effective unit weight of the three clay layers are s_{u1} , s_{u2} , s_{u3} and γ'_1 , γ'_2 , γ'_3 respectively. The thicknesses of the top and middle layers are finite as t_1 and t_2 , where the thickness of the bottom (3rd) layer with thickness of t_3 (nominally) infinite. The standard cone penetrometer with $D = 0.0357$ m and cone tip angle = 60° was considered. The cone/soil interface was simulated using elasto-plastic nodal joint elements. A rigid cone with displacement control was set up for large penetration analysis. The cone/soil interface (limiting) shear strength was taken as αs_u .

The soil parameters were selected based on an extensive survey on existing offshore geotechnical characterization reports. The three-layer geometries considered here are commonly encountered in the Gulf of Thailand and Sunda Shelf, including Java Sea, as reported by [Handidjaja et al. \(2004\)](#). These suggest a typical stiff layer undrained shear strength of 40~100 kPa and minimum soft layer strength of ~10 kPa, with the stiff to soft clay strength

ratio lying in the range of 2~8. For some clay layers strength increase with depth with a gradient of $k_s = 0 \sim 3$ kPa/m. In this study, the undrained shear strength values were selected, as listed in Tables 1~3, from these ranges of practical interests. The roughness factor α of the standard cone used in the field lies in the range of 0.2-0.6 (Lu et al. 2004), thus values of $\alpha = 0 \sim 0.3$ were considered in this study. The *in situ* stress ratio Δ varied from 0 to 0.75, while the soil strength non-homogeneity factor $k_s D/s_{um}$ in the range of 0 to 0.21 (taking $k_s = 0 \sim 3$ kPa/m). Both thin stiff and soft clay layers embedding in a uniform or a NC clay deposit were considered. The soil strength ratios between two successive layers and thickness ratios of the middle thin layer (t_2/D) were also varied to study their effects. The effects of changing s_u value for individual clay layer as well as among layers were discussed by Ma et al. (2017).

Analysis Details

Due to continuous deep penetration of the cone penetrometer, LDFE analyses were performed using the RITSS (Remeshing and Interpolation Technique with Small Strain; Hu and Randolph 1998a) method. In LDFE/RITSS analysis, a series of small strain incremental analyses are combined with fully automatic adaptive mesh generation of the entire domain. After each mesh generation, all field variables, such as stresses and material properties are interpolated from the old mesh to the new mesh. The small strain analysis between each mesh generation is conducted using the finite element package AFENA (Carter and Balaam 1995) developed at the University of Sydney. The *H*-adaptive mesh refinement cycles (Hu and Randolph 1998b) in each mesh generation is to optimize the mesh based on minimizing strain-errors. The displacement increment size and the number of small strain incremental analyses between each re-meshing are chosen such that the cumulative penetration between re-meshing stages remains in the small strain range and is less than half the minimum element size. Due to the steady and

practical nature of the LDFE/RITSS method, the cone penetration is simulated from the seabed surface using the displacement control technique.

The soil domain is axisymmetric with $100D$ in radius and $100D$ in depth to eliminate domain boundary effects on cone analysis results. Both vertical sides (outer side and soil/vertical axis) are set up as roller condition with no-horizontal movements, but free vertical movements. The bottom boundary is set up as hinge condition without any movement allowed. In all FE analyses, six-noded triangular elements with three internal Gauss points are used. Figure 3 displays a typical mesh for the cone penetration in a three-layer stiff-soft-stiff clay deposit, at different penetration stages. The finer mesh around the cone tip is generated to ensure the accuracy of the computed results.

The soil is modelled as a linear elastic-perfectly plastic material obeying a Tresca yield criterion. Considering the relatively fast penetration of the field cone penetrometer (20 mm/s), all the analyses simulate undrained conditions with a Poisson's ratio $\nu = 0.49$ (sufficiently high to give minimal volumetric strains, while maintaining numerical stability) and friction and dilation angles $\phi = \psi = 0$ in total stress analysis. The *in situ* lateral stress coefficient K_0 has been set as 1 in most of the analyses (Table 1 and Table 3), except those in Table 2 where the effect of *in situ* stress ratio was investigated. First series of analyses are carried out varying the strength ratio of s_{u1}/s_{u2} or s_{u3}/s_{u2} (identical parameters for 1st and 3rd layers) and the layer thickness ratios of t_1/D and t_2/D , but with cone tip roughness factor $\alpha = 0.3$, as tabulated in Table 1. Second series of analyses are conducted varying the *in situ* stress ratio Δ , for a thin interbedded uniform layer (soft and stiff) in NC clay, as listed in Table 2. In the third series of analyses, different strength non-homogeneity factors $k_s D/s_{um}$ have been assumed, with a thin uniform layer (both soft and stiff) sandwiched by NC clays (Table 3). As the effect of the soil rigidity index I_r has been systematically explored by [Ma et al. \(2016, 2017\)](#), a constant rigidity index of $I_r = 167$ is used in this study.

189

190 **Results and Discussion**

191 ***Effect of roughness factor***

192 **Effect of roughness factor in soft-stiff-soft clays**

193 To examine the effect of cone tip roughness factor on the resulting penetration resistance in
194 soft-stiff-soft clays, two groups (Groups AI and AII; Table 1) of analyses have been performed
195 considering $\alpha = 0.3$. Figure 1b displays the issue studied here schematically. Figure 4a plots
196 the result of Group AI with various 2nd layer thickness ratios ($t_2/D = 2, 5, 10, 15$), but with
197 identical strength ratio of $s_{u2}/s_{u1} = s_{u2}/s_{u3} = 4$. The strength ratio effects are shown in Figure 4b
198 with identical layer thicknesses and various strength ratios of $s_{u2}/s_{u1} = s_{u2}/s_{u3} = 2, 4$ and 6. The
199 soil failure mechanisms are displayed in Figure 4c ($t_1/D = 16.8, t_2/D = 20, s_{u2}/s_{u1} = s_{u2}/s_{u3} = 4$).

200 In Figure 4a, the cone resistance profiles follow a similar trend of that of a smooth cone. A
201 unique distance from the kink in the 1st layer to the interface ($\Delta_1 = 0.8D$) can be found for
202 identifying the interface between 1st and 2nd (i.e. soft to stiff) layers, regardless of the 2nd layer
203 thickness ratio t_2/D . This value of $\Delta_1 = 0.8D$ is identical to that identified for $\alpha = 0$ (Ma et al.
204 2015). For identifying the interface between the 2nd and 3rd (i.e. stiff to soft) layers, the interface
205 location can be found uniquely at $1.7D$ (referred to as Δ_2) above the kink in the cone resistance
206 profile in the 3rd (soft) layer, whereby the corresponding value for $\alpha = 0$ is $\Delta_2 = 1.3D$. This
207 increase is possibly due to the 2nd layer of stiff clay (and hence the 2nd - 3rd layer interface)
208 being tailed locally with the partially rough cone tip to a deeper depth in the 3rd layer, as shown
209 in Figure 4c(i). The stiff soil in the 2nd layer is still attached to the partially rough cone ($\alpha =$
210 0.3) shoulder at a penetration depth of $d/D = 38.2$ (Figure 4c(i)), whilst that detached from the
211 smooth cone ($\alpha = 0$) shoulder at around $d/D = 38.1$ (Figure 4c(ii)).

Figure 4a also illustrates a comparison of the penetration resistance for $\alpha = 0.3$ and $\alpha = 0$, with $t_2/D = 5$. It is clear that the cone resistance for $\alpha = 0.3$ is slightly higher than that for $\alpha = 0$, and the difference is within 5%. However since the limiting penetration resistance for $\alpha = 0.3$ in single layer uniform clay (Equation 3) is also higher, the resultant effect on correction factor k becomes minimal (i.e. $k = q_{\text{net},f}/q_{\text{net},m}$, with both $q_{\text{net},f}$ and $q_{\text{net},m}$ being increased due to the increased α , resultant ratio of k is kept relatively unchanged). For instance, (i) for $t_2/D = 2$ and $\alpha = 0.3$, the correction factor is 1.63 compared to 1.64 for $\alpha = 0$ (i.e. the difference is 0.01); (ii) for $t_2/D = 5$ and $\alpha = 0.3$, the correction factor becomes 1.25, which is the same as the finding for $\alpha = 0$. Similar conclusions can be drawn for $t_2/D = 10$ and $t_2/D = 15$, where the variation of the correction factor is within 0.01 (i.e. $< 1\%$). Thus the correction factor is not affected by α .

Figure 4b illustrates the combined effect of α and various strength ratios of $s_{u1}/s_{u2} = 2, 4$ and 6 . Again, the 1st-2nd layer (i.e., soft-stiff) interface can be identified at $\Delta_1 = 0.8D$ below the kink in the 1st layer profile, and the 2nd -3rd layer interface can be detected at $1.7D$ above the kink in the 3rd layer profile. In addition, the cone resistance correction factors for $\alpha = 0.3$ and 0 are very close, with a maximum difference being 0.02 (i.e. $< 2\%$), as shown in Figure 4b.

In summary, the framework proposed by Ma et al. (2015) for layer identification and interpretation using the smooth cone penetrometer data are also applicable for the cone with $\alpha = 0.3$, where the ultimate bearing factor N_{kt} and Δ_2 need to be updated for $\alpha = 0.3$ and the cone resistance correction factor k for the thin stiff layer can be kept the same.

Effect of roughness factor in stiff-soft-stiff clays

The analyses for $\alpha = 0.3$ have been performed to examine the effect of α on the cone penetration resistance profile in stiff-soft-stiff clays ($s_{u1}/s_{u2} = s_{u3}/s_{u2} = 4, 6, 8$; $t_2/D = 4, 12, 20$; $t_1/D = 20$;

235 $I_{r1} = I_{r2} = I_{r3} = 167$; Groups AIII and AIV, Table 1). Figure 1c displays the issue studied here
236 schematically. The results are presented in Figure 5a and Figure 5b.

237 Figures 5a and 5b display the effects of middle layer thickness and layer strength ratio on the
238 cone resistance profiles with $\alpha = 0.3$. The layer interface identification can be performed using
239 the demarcation distances found in the last section: $\Delta_1 = 0.8D$ for the interface from soft to stiff
240 clays; $\Delta_2 = 1.7D$ for the interface from stiff to soft clays, with both being consistent with Figure
241 4. One penetration resistance profile for $\alpha = 0$ is also included in each figure, which shows
242 increasing cone penetration resistance ($\sim 5\%$) with increasing α from 0 to 0.3. However, with
243 the same reasoning in the previous section as the both full resistance in a single clay layer ($q_{\text{net},f}$)
244 and measured resistance in the thin layer ($q_{\text{net},m}$) are increased in the same proportion, the
245 correction factor ($k = q_{\text{net},f}/q_{\text{net},m}$) for resistance in the middle clay layer is not affected. For
246 instance, (i) for $t_2/D = 4$ and $\alpha = 0.3$, the correction factor is 0.81 compared to 0.82 for $\alpha = 0$
247 (i.e. the difference is 0.01; see Figures 5a and 5b); (ii) for $t_2/D \geq 12$ and $\alpha = 0.3$, the correction
248 factor becomes 1.0, which is the same as the findings for $\alpha = 0$.

249 In summary, the updates proposed in the previous section for a partially rough cone ($\alpha = 0.3$)
250 penetration in soft-stiff-soft soils can be used as updates for a partially rough cone ($\alpha = 0.3$)
251 penetration in stiff-soft-stiff soils. These updates can be combined with the framework
252 proposed for a smooth cone ($\alpha = 0$) in stiff-soft-stiff soils as expressed in Equation 4 and shown
253 in Figure 2.

254 In order to propose a general relationship between Δ_2 and α , additional analyses have been
255 carried out for $\alpha = 0.1$ and 0.2. While Δ_1/D remains the same (0.8) for all values of α , Δ_2/D
256 increases with increasing α , with a relationship given in Figure 6. Additional analyses have
257 confirmed these distances for strength ratios close to 1 e.g. 1.1.

In addition, in the above sections, the thickness of the 2nd layer has been considered as $> 2D$, it is worthwhile to investigate the applicability of the proposed solutions for a very thin embedded layer with $t_2/D \leq 2D$. For an interbedded stiff layer with less than $2D$ (0.07 m) thickness, the overall effect is positive and very minimal in engineering practice. For an interbedded soft layer $t_2/D \leq 2D$, further investigation is required as identified $\Delta_1 + \Delta_2 = 2 \sim 2.5D$ (depending on α) $\geq 2D$. A group of analyses were therefore performed for $t_2/D = 2D$ and $1D$, and varying strength ratios and cone tip roughness factors ($s_{u1}/s_{u2} = s_{u3}/s_{u2} = 4, 8$; $\alpha = 0, 0.3$). The results are plotted in Figure 7. It can be seen that, in the 2nd (soft) layer, there is a single (and sharp) kink or lowest point on the penetration resistance profiles for $t_2/D = 1$ and 2 , unlike two kinks on those for $t_2/D \geq 4$. Conversely, two kinks with a transitional stage (gently increase part after the lowest point) on the cone penetration resistance profile in the 2nd (soft) layer indicate $t_2/D \geq 4$, a single sharp lowest point indicates $t_2/D \leq 2$. Guidelines for layer identification can be summarized as

$$\begin{cases} \Delta_1 = 0.8D, \Delta_2 = 1.2D & \text{for } t_2/D = 2, \alpha = 0 \\ \Delta_1 = 0.7D, \Delta_2 = 1.3D & \text{for } t_2/D = 2, \alpha = 0.3 \\ \Delta_1 = 0.6D, \Delta_2 = 0.4D & \text{for } t_2/D = 1, \alpha = 0 \text{ and } 0.3 \end{cases} \quad (5)$$

In terms of strength interpretation i.e. correction factor, k , a conservative value is suggested to be considered. For $t_2/D \leq 2$, the value of $k = 0.125$, which is the value for $t_2/D = 0$ and $s_{u, \text{stiff}} / s_{u, \text{soft}} = 8$, should be used.

Effect of *in situ* stress ratio

Effect of *in situ* stress ratio in single layer clay

To explore the effect of the *in situ* stress ratio, $\Delta = (\sigma_{v0} - \sigma_{h0})/2s_u$, on deep bearing capacity factor, the results of various stress ratios $\Delta = 0, 0.25, 0.5, 0.75$ are plotted in Figure 8. Note, for

279 achieving constant $\Delta = \frac{\sigma_{v0} - \sigma_{h0}}{2s_u} = \frac{(1-K_0)\sigma_{v0}}{2s_u} = \frac{(1-K_0)\gamma'd}{2(s_{um} + k_s d)}$ over full penetration depths, it was
 280 necessary to consider clay strength increasing with depth.

281 Figure 8a shows the results in terms of cone net penetration resistance, normalized by the local
 282 shear strength $s_{u,local} = s_{um} + k_s d$ as strength increases with depth, as a function of normalized
 283 penetration depth for a constant $I_r = 167$, $k_s D/s_{um} = 2.14$ (Group BI; Table 2). The normalized
 284 resistance decreases with increasing Δ . The limiting bearing factors i.e. N_{kt} values were picked
 285 averaging the stable part of each penetration resistance profile ($d/D > 9$), and plotted in Figure
 286 8b. The N_{kt} values from previous studies (Teh and Houslby 1991; Yu 2000; Lu et al. 2004;
 287 Walker and Yu 2006; Liyanapathirana 2009) are also included in Figure 8b for comparison.
 288 The current results display a consistent trend to the previous results, staying right in between
 289 the upper bound by Lu et al. (2004) and the lower bound by Liyanapathirana (2009). Adding
 290 the effect of *in situ* stress ratio on cone limiting bearing factor, Equation 2 can be extended to

$$291 \quad N_{kt} = 3.47 + 1.56 \ln I_r + 1.3\alpha - 1.85\Delta \quad (6)$$

292 **Effect of *in situ* stress ratio in layered soil with a stiff layer interbedded in NC clay**

293 To examine the effect of the *in situ* stress ratio on the resulting penetration resistance in layered
 294 soils, the layered soil is set up as a stiff clay layer interbedded in a NC clay. The *in situ* stress
 295 ratio Δ varies from 0 to 0.75, with identical strength ratio $s_{u2}/s_{u1b} = 4$, layers thickness ratios of
 296 $t_1/D = 16.8$, $t_2/D = 5$, cone tip roughness factor $\alpha = 0$ (Group BII; Table 2). The problem is
 297 schematically illustrated in Figure 1d. Figure 9 displays the penetration resistance profiles
 298 normalized by the local shear strength, $s_{u,local}$, for NC clay layers. For the 2nd uniform stiff layer,
 299 the cone resistance is normalized by the shear strength at the bottom of 1st layer, $s_{u1b} = s_{um} +$
 300 $k_s t_1$ to be consistent with Ma et al. (2015, 2017), where the cone resistance of the stiff layer was
 301 normalized by the shear strength of the soft layer. Clearly, the three curves start separating

from the 1st layer, and the normalized resistance increases with decreasing Δ . From Figure 9, it can be seen that the 1st-2nd layer interface can be detected at a unique distance ($\Delta_1 = 0.8D$) below the kink in the 1st layer resistance profile, regardless of the *in situ* stress ratio. Similarly, the 2nd-3rd layer interface can be identified at an identical distance ($\Delta_2 = 1.3D$) above the kink in the 3rd layer. These values are consistent with those for $\Delta = 0$.

Also, in general the soil flow mechanisms at corresponding depths relative to the thicknesses of the layers are very similar, comparing to those for $\alpha = 0$, $\Delta = 0$.

Figure 9 also illustrates a comparison of the penetration resistance for $\Delta = 0.75$, 0.5 and 0, while $t_2/D = 5$, $s_{u2}/s_{u1b} = 4$. It is clear that the resistance for $\Delta = 0$ is slightly larger than that for $\Delta = 0.5$, and both of them are larger than that for $\Delta = 0.75$. However since the penetration resistance of the cone with $\Delta = 0.75$ and $\Delta = 0.5$ in single layer uniform clay (Figure 8) is also lower (Equation 6), the resultant effect on correction factor k becomes minimal. To be more specific, (i) the correction factor is 1.17 for $\Delta = 0.75$ and 1.13 for $\Delta = 0$ (i.e. the difference is 0.04 or 3.6%); (ii) the correction factor is 1.15 for $\Delta = 0.5$, and the variation of k value is 0.02 (or 1.8%). Therefore the correction factors for various *in situ* stress ratios can be taken reasonably as similar.

In summary, the effect of *in situ* stress ratio in soft-stiff-soft clays is minimal, certifying the applicability of the interpretation framework established by Ma et al. (2015) for $\Delta = 0$ (Equation 4) for any values of Δ with reasonable accuracy.

Effect of *in situ* stress ratio in layered soil with a soft layer interbedded in NC clay

A similar investigation was also carried out for cone penetration in a layered soil with a soft layer interbedded in a NC clay. The *in situ* stress ratio varies as $\Delta = 0$, 0.5, 0.75, but with identical strength ratio $s_{u1b}/s_{u2} = 4$, layers thickness ratio $t_1/D = 16.8$, $t_2/D = 5$, cone tip roughness factor $\alpha = 0$ (Group BIII; Table 2). The problem is schematically illustrated in Figure

1e. The corresponding penetration resistance profiles are normalized by the local shear strength $s_{u, \text{local}} = s_{\text{um}} + k_s \cdot d$ in the NC clay layers. In the 2nd uniform soft layer, the cone resistance is normalized by the shear strength at the bottom of 1st layer, $s_{u1b} = s_{\text{um}} + k_s \cdot t_1$. The cone bearing factors are plotted in Figure 10 as a function of penetration depth. From Figure 10, it is clear that the 1st-2nd (i.e., stiff-soft) layer interface can be detected at a unique distance ($\Delta_2 = 1.3D$) below the kink in 1st layer profile, regardless of the *in situ* stress ratio. Likewise, the 2nd-3rd (soft-stiff) layer interface can be identified at an identical distance ($\Delta_1 = 0.8D$) above the kink in the 3rd layer. These values coincide with those for $\Delta = 0$, as reported by Ma et al. (2017). Again, for this interbedded soft deposits and $\alpha = 0$, $\Delta = 0$ ($s_{u2}/s_{u1} = s_{u2}/s_{u3} = 4$ or $s_{u1b}/s_{u2} = 4$), the soil failure mechanisms at corresponding depths relative to the thicknesses of the layers are very similar in general.

Figure 10 also illustrates a comparison of the penetration resistance for $\Delta = 0.75$, 0.5 and 0, while $t_2/D = 5$, $s_{u1b}/s_{u2} = 4$. Obviously the resistance for $\Delta = 0 >$ for $\Delta = 0.5 >$ for $\Delta = 0.75$. The penetration resistance profiles in single layer clay follow consistent trend (Equation 6 and Figure 8), resulting in very minimal effect on correction factor k . For instance, for $\Delta = 0.75$, the correction factor is 0.75 compared to 0.77 for $\Delta = 0$ (i.e. the difference is 0.02 or 2.7%). For $\Delta = 0.5$, the correction factor is 0.75, which is the same as the one for $\Delta = 0$. Thus the variation of the correction factor is within 0.02, and it can be considered as negligible.

In summary, the effect of *in situ* stress ratio on identification of layer boundaries and resistance correction factor is minimal, confirming the validity of the framework (Equation 4, Figure 2) developed by Ma et al. (2017) regardless of $\Delta = 0$.

Cone penetration test in a thin layer embedded in NC clay

In previous studies, except for exploring the effect of Δ , the three-layer soil profiles consist of uniform clays all through the three layers. In the following sub-sections, the performance of

the standard cone penetrometer in a thin layer (either strong or soft) sandwiched in a backbone NC clay deposit is investigated.

Cone penetration in a stiff layer embedded in NC clay

Figure 11 plots the results of the smooth-tip cone penetration in a uniform stiff clay layer ($s_{u2}/s_{u1b} = 4$, $t_2/D = 10$) embedded in a NC clay deposit ($s_{um} = 0.5$, $k_s D/s_{um} = 0.21$, $t_1/D = 30$, $\Delta = 0$; Group CI in Table 3). The problem is schematically shown in Figure 1d as well. The relatively small diameter of the standard cone ($D = 37.5\text{mm}$) induces a low degree of soil strength non-homogeneity of $k_s D/s_{um} = 0.21$, which is close to that of uniform clay of $k_s D/s_{um} = 0.21$. It can be seen that the profiles for $k_s D/s_{um} = 0$ and 0.21 overlap each other, indicating the effect of non-homogeneity is minimal for the smooth-tip cone (as was also found from the analyses on single layer clay; Ma et al. 2014).

Cone penetration in a soft layer embedded in NC clay

Figure 12 plots the results of the smooth-tip cone penetration in a uniform soft layer ($s_{u1b}/s_{u2} = 4$, $t_2/D = 10$) embedded in a NC clay deposit with ($s_{um} = 0.5$, $k_s D/s_{um} = 0.21$, $t_1/D = 30$, $\Delta = 0$; Group CII in Table 3). The problem is schematically shown in Figure 1e. Similar to a strong layer embedded in a NC clay deposit, the curves for $k_s D/s_{um} = 0$ and 0.21 are consistent, indicating minimal influence of soil strength non-homogeneity. Due to the small diameter of the standard cone (i.e. $D = 37.5\text{ mm}$), the range studied here of $k_s D/s_{um} = 0$ and 0.21 covers the practical range of NC clays in the field.

In summary, the formulas and equations established for uniform clays are applicable for interpreting cone penetrometer (with fully smooth tip) data in NC clay deposits with an interbedded uniform clay layer.

Concluding Remarks

In this paper, the effects of cone tip roughness factor α , *in situ* stress ratio Δ , and soil strength non-homogeneity factor $k_s D/s_{um}$ on cone penetration response (and hence bearing factor) in soft-stiff-soft and stiff-soft-stiff clays were investigated systematically varying various relevant parameters. Based on the design framework proposed for a cone with tip roughness factor $\alpha = 0$ in three uniform clay layers by Ma et al. (2017), as expressed in Equation 4 and shown in Figure 2, some conclusions can be drawn as listed below.

- In general, the design framework developed for $\alpha = 0$ can be used for $\alpha = 0.3$ in three-layer uniform and non-uniform clays.
- The cone resistance correction factors in the middle thin layer were not affected by α , Δ and $k_s D/s_{um}$.
- The layer interface identification was not affected by Δ and $k_s D/s_{um}$, but by α .
- For $t_2/D > 2$ and for the range of investigated α of 0~0.3, the layer interface from soft to stiff could be identified at a distance of $\Delta_1/D = 0.8$ regardless of α . However, the distance of the layer interface from stiff to soft, Δ_2/D , increased with increasing α (Figure 6).
- For $t_2/D \leq 2$ and for interbedded soft layer, Δ_1/D and Δ_2/D were identified as: for $t_2/D = 2$, $\alpha = 0$, the distances are ($\Delta_1 = 0.8D$, $\Delta_2 = 1.2D$); for $t_2/D = 2$, $\alpha = 0.3$, the distances are ($\Delta_1 = 0.7D$, $\Delta_2 = 1.3D$); for $t_2/D = 1$, $\alpha = 0$ and 0.3, the distances are ($\Delta_1 = 0.6D$, $\Delta_2 = 0.4D$). Overall influence of a very thin ($t_2/D \leq 2$) interbedded stiff layer is minimal.
- For $t_2/D \leq 2$ and for interbedded soft layer, in terms of strength interpretation i.e. correction factor, k , a conservative value of $k = 0.125$ could be used.

It should be noted that, when the design framework is used for $\alpha > 0$ with different Δ and $k_s D/s_{um}$, the corresponding full limiting cone resistance (or bearing factor) in single layer of clay should be used in the design formulas proposed.

Although the numerical results have been validated against existing published data of cone penetration in layered clays, mainly large deformation FE results, more validations against physical testing data and field test data might be needed for the application in field data interpretations.

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410 **Notation**

411	α	soil-cone interface friction ratio
412	D	diameter of cone penetrometer
413	d	penetration depth of cone shoulder
414	d_p	depth of peak resistance in 1 st layer
415	d_{rem}	depth in 3 rd layer required to mobilize stable resistance
416	E	Young's modulus
417	f_s	sleeve friction
418	G	shear modulus
419	I_r	rigidity index
420	$I_{r,stiff}$	rigidity index of stiff layer
421	$I_{r,soft}$	rigidity index of soft layer
422	k	resistance correction factor in the thin embedded layer
423	K_0	<i>in situ</i> lateral Earth pressure coefficient at rest
424	k_s	strength gradient
425	k	resistance correction factor in an interbedded thin layer
426	N_{kt}	bearing capacity factor of cone penetrometer
427	q_c	measured cone resistance
428	q_t	total cone resistance
429	q_{net}	net cone resistance
430	$q_{net,f}$	full net resistance
431	$q_{net,m}$	measured net resistance
432	$q_{net1,f}, q_{net2,f}, q_{net3,f}$	full net resistance of 1 st , 2 nd , 3 rd layer

433	$q_{\text{net1,m}}, q_{\text{net2,m}}, q_{\text{net3,m}}$	measured net resistance of 1 st , 2 nd , 3 rd layer
434	s_u	undrained shear strength
435	s_{u1}, s_{u2}, s_{u3}	undrained shear strength of 1 st , 2 nd , 3 rd layer
436	$s_{u,\text{stiff}}$	undrained shear strength of stiff layer
437	$s_{u,\text{soft}}$	undrained shear strength of soft layer
438	t_1, t_2, t_3	thickness of 1 st , 2 nd , 3 rd layer
439	γ'	soil effective unit weight
440	$\gamma'_1, \gamma'_2, \gamma'_3$	soil effective unit weight of 1 st , 2 nd , 3 rd layer
441	σ_{h0}	in situ total horizontal stress
442	σ_{v0}	in situ total vertical stress
443	Δ	initial stress anisotropy = $(\sigma_{v0} - \sigma_{h0})/2s_u$
444	Δ_1	distance from kink (sharp rise point) to soft-stiff layer interface
445	Δ_2	distance from stiff-soft layer interface to the merging point
446	ν	Possion's ratio
447		

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Table 1 LDFE analyses performed to study the effect of cone tip roughness factor

Analyses	layer 1		layer 2			layer 3		Remarks
	t_1/D	S_{u1} (kPa)	t_2/D	S_{u2} (kPa)	$S_{u2}/S_{u1} = S_{u2}/S_{u3}$	t_3/D	S_{u3} (kPa)	
Group AI	16.8	10	2, 5, 10, 15	40	4	∞	10	Soft-stiff-soft
Group AII	16.8	10	5	20, 40, 60	2, 4, 6	∞	10	
Group AIII	20	40	4, 12, 20	10	1/4	∞	40	Stiff-soft-stiff
Group AIV	20	40, 60, 80	4	10	1/4, 1/6, 1/8	∞	40, 60, 80	

For all the cases: $\gamma'_1 = \gamma'_2 = \gamma'_3 = 6 \text{ kN/m}^3$; $l_1 = l_2 = l_3 = 167$; $\alpha = 0, 0.1, 0.2, 0.3$ (for tip only while 0 for shaft); $D = 0.0357 \text{ m}$; penetrating from soil surface

Table 2 LDFE analyses performed to study the effect of *in situ* stress ratio

Analyses	layer 1				layer 2			layer 3				Remarks
	t_1/D	S_{u1} (kPa)	k_{s1}	Δ	t_2/D	S_{u2} (kPa)	Δ	t_3/D	S_{u3} (kPa)	k_{s1}	Δ	
Group BI	∞	0.05	3	0, 0.25, 0.5, 0.75	--							Single layer
Group BII	16.8	0.05	3	0, 0.5, 0.75	5	7.4	0	∞	0.05	3	0, 0.5, 0.75	Soft-stiff-soft
Group BIII	20	0.05	3	0, 0.5, 0.75	4	0.55	0	∞	0.05	3	0, 0.5, 0.75	Stiff-soft-stiff

For all the cases: $\gamma'_1 = \gamma'_2 = \gamma'_3 = 6 \text{ kN/m}^3$; $l_1 = l_2 = l_3 = 167$; $\alpha = 0$ (for both shaft and tip); $D = 0.0357 \text{ m}$; penetrating from soil surface

Figures

Figure 1. Study plan of cone penetration in various three-layer deposits

Figure 2. Design chart of correction factor k for soft-stiff-soft and stiff-soft-stiff clays ($\alpha = 0$; from Ma et al., 2017)

(a) Initial mesh (axes normalized by cone diameter D)

(b) Finer mesh around cone with deep penetration

Figure 3. Typical mesh at different stages for cone penetration in three-layer clay

(a) Penetration resistance profiles: $t_1/D = 16.8$, $t_2/D = 2, 5, 10, 20$, $s_{u2}/s_{u1} = s_{u2}/s_{u3} = 4$

(b) Penetration resistance profiles: $t_1/D = 16.8$, $t_2/D = 5$, $s_{u2}/s_{u1} = s_{u2}/s_{u3} = 2, 4, 6$

(c) Comparison of soil flow mechanisms: $t_1/D = 16.8, t_2/D = 20, s_{u2}/s_{u1} = s_{u2}/s_{u3} = 4$

Figure 4. Effect of cone tip roughness factor α on cone penetration resistance profiles in soft-stiff-soft clay deposits ($l_{r1} = l_{r2} = l_{r3} = 167$; Groups AI and AII, Table 1)

(a) Effect of middle layer thickness: $t_1/D = 20$, $t_2/D = 4, 12, 20$, $s_{u1}/s_{u2} = s_{u3}/s_{u2} = 4$

(b) Effect of strength ratio: $t_1/D = 20$, $t_2/D = 4$, $s_{u1}/s_{u2} = s_{u3}/s_{u2} = 4, 6, 8$

Figure 5. Effect of cone tip roughness factor α on cone penetration in stiff-soft-stiff clay deposits ($I_{r1} = I_{r2} = I_{r3} = 167$; Groups AIII and AIV, Table 1)

Figure 6. Distance from kink in soft layer to stiff-soft layer interface, Δ_2 , as a function of α

(a) $t_2/D = 2$

(b) $t_2/D = 1$

Figure 7. Layer identification distances for an extremely thin soft mid-layer

(a) Effect of *in situ* stress ratio on cone factor N_{kt}

(b) Comparison of stabilised bearing factor N_{kt} with previous solutions

Figure 8. Effect of *in situ* stress ratio Δ in single layer clay

(Group BI, Table 2)

*For 2nd layer, the resistance is normalised by s_{u1b}

Figure 9. Effect of *in situ* stress ratio Δ on cone penetration resistance in soft-stiff-soft clay deposits ($t_1/D = 16.8$, $t_2/D = 5$, $s_{u2}/s_{u1b} = 4$; Group BII, Table 2)

*For 2nd layer, the resistance is normalised by s_{u1b}

Figure 10. Effect of *in situ* stress ratio Δ on cone penetration resistance profiles in stiff-soft-stiff clay deposits ($t_1/D = 20$, $t_2/D = 4$, $s_{u1b}/s_{u2} = 4$; Group BIII, Table 2)

*For 2nd layer, the resistance is normalised by s_{u1b}

Figure 11. Effect of top and bottom layer strength non-homogeneity on cone penetration resistance in NC clay with an interbedded stiff clay layer ($t_1/D = 30$, $t_2/D = 10$, $s_{u2}/s_{u1b} = 4$; Group CI, Table 3)

*For 2nd layer, the resistance is normalised by s_{u1b}

Figure 12. Effect of top and bottom layer strength non-homogeneity on cone penetration resistance in NC clay with an interbedded soft clay layer ($t_1/D = 30$, $t_2/D = 10$, $s_{u2}/s_{u1b} = 1/4$; Group CII, Table 3)

Table 3 LDFE analyses performed to study the effect of strength non-homogeneity

Analyses	layer 1			layer 2			layer 3			Remarks
	t_1/D	S_{um1} (kPa)	k_{s1}	t_2/D	S_{u2} (kPa)	S_{u2}/S_{u1b}	t_3/D	S_{um3} (kPa)	k_{s1}	
Group CI	30	0.5	3	10	15	4	∞	0.5	3	Soft-stiff-soft
Group CII	30	0.5	3	10	0.94	1/4	∞	0.5	3	Stiff-soft-stiff

For all the cases: $\gamma'_1 = \gamma'_2 = \gamma'_3 = 6 \text{ kN/m}^3$; $\Delta = 0$; $l_{r1} = l_{r2} = l_{r3} = 167$; $\alpha = 0$ (for both shaft and tip); $D = 0.0357 \text{ m}$; penetrating from soil surface