Interpretation of Layer Boundaries and Shear Strengths for Soft-Stiff-Soft Clays Using CPT Data: LDFE Analyses

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Abstract: This paper describes a new approach for interpreting cone penetrometer data in soft-stiff-soft clay deposits. The identification of layer boundaries and interpretation of shear strength profile were of particular interest. The proposed approach was based on an extensive parametric study using large deformation finite element (LDFE) analyses, with a standard cone penetrometer penetrated continuously from the soil surface. The LDFE model has been validated against existing theoretical solutions and numerical results, with good agreement obtained. Regardless of strength ratio between two successive layers, the interface of soft-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-soft layers can be demarked at $1.3D$ above the kink in penetration resistance profile in the bottom soft layer. The undrained shear strength of a soft clay layer can be interpreted using a single layer approach and the resistance profile without the influence of the adjacent stiff layer. The interpretation for the interbedded stiff layer necessitates implementing a correction factor, which is shown to be a function of the

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17 thickness and rigidity index of the stiff layer and the strength ratio between that layer and the
18 bottom layer. The proposed design framework is illustrated through a flow chart to be used in
19 practice.

20 **CE Database subject headings:** Clays; Cone penetration tests; Flow patterns; Layered
21 soils; Numerical analysis; Offshore structures; Penetration resistance; Shear strength; Site
22 investigation.

Introduction

Background

Depletion of known reserves in the shallow waters of traditional hydrocarbon regions is resulting in exploration in deeper, unexplored and undeveloped environments. These are exhibiting more complex soil conditions at the seabed. In emerging provinces and fields, highly layered soils are prevalent. For instance, over 75 % of the case study data sets forming the basis for the InSafeJIP (2010) involved stratified seabed profiles, with interbedded layers of clay and sand displaying strong variations in shear strength. The Sunda Shelf, offshore Malaysia, Australia's Bass Strait and North-West Shelf, Gulf of Thailand, South China Sea, Caspian Sea, offshore India and Arabian Gulf are particularly problematic in terms of stratigraphy and soil types. Layered deposits are also encountered in the Gulf of Mexico and Gulf of Guinea (Menzies and Roper 2008; Colliat and Colliard 2011; Menzies and Lopez 2011). This paper focuses on characterising three layer soft-stiff-soft clay deposits.

The proper characterisation of soil layering is essential for predicting offshore foundations/anchoring systems response profile. This is because (i) the thicknesses of the layers relative to the foundation/anchor diameter and (ii) the strength ratios of two successive layers significantly influence the evolving soil failure mechanisms and, as a result, the entire behaviour of the foundation/anchoring systems. Typical examples include the likelihood and severity of punch-through failure for spudcan foundations (Hossain et al. 2010a, b; 2011; Lee et al. 2013; Hossain 2014), the depth and thickness of an interbedded strong layer and hence the required length for skirted foundations and suction caisson anchors (Watson et al. 2006; Alessandrini and Lebois 2007; Thomas and Kergustanc 2007).

It is now a general consensus that prior to any offshore project, site-specific geotechnical investigation is required to carry out through continuous borehole sampling combined with

continuous penetrometer tests. The primary goals of this testing are layer identification and determination of strength parameters for each layer. Comparing with laboratory tests, in situ penetrometer tests provide nearly continuous data and hence allow for avoiding high degree of subjectivity in layer demarcation and strength interpretation particularly where some degree of data scatter is present (Hossain et al. 2012). The difficulty of obtaining high-quality soil samples (for laboratory testing) from emerging provinces and fields has also placed increased reliance on in situ testing data.

Cone Penetration Test (CPT)

Currently, the most commonly adopted in situ test in offshore site investigations is the cone penetration test (CPT). This test has a strong theoretical background. The standard cone penetrometer is cylindrical in shape with a conical tip that has a base area of 10 cm² (diameter $D = 35.7$ mm) and a 60° tip-apex angle, as shown schematically in Figure 1. Parameters measured during a piezocone test include (a) cone tip resistance, q_c ; (b) sleeve friction, f_s ; and (c) pore water pressure, u_2 . This paper considers only cone tip resistance.

The penetration tests are carried out at a rate of $v = 20$ mm/s. For clay deposits, the velocity is fast enough to ensure undrained conditions. As such, the cone factor N_{kt} is used to relate the net cone tip resistance q_{net} to the intact undrained shear strength s_u as

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

where q_t is the total cone tip resistance and σ_{v0} is the total overburden stress at the level of the cone shoulder, d (Figure 1). For this study, the correction for the effect of unequal pore pressure was not necessary as the cone was simulated as a solid shaft (see Figure 1; i.e. $q_t =$ measured tip resistance q_c). The undrained shear strength for each layer deduced using Equation 1 may not represent the actual value as the layer thickness may not be sufficient for allowing the cone

penetrometer to mobilize the stable (full) penetration resistance for that layer. A corresponding adjustment is required to interpret the actual shear strength. This is addressed in this paper.

Previous Work

A number of investigations have been carried out on cone penetration through analytical, numerical and experimental work, although mostly limited to response in single layer clay with uniform undrained shear strength. The bearing capacity factor of cone has been explored largely through 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 2000). Recently, this problem has been addressed through large deformation finite element (LDFE) analysis and centrifuge model test (van den Berg 1994; Bolton et al. 1999; Lu et al. 2004; Walker and Yu 2006; Liyanapathirana 2009). The results will be discussed later in relation to the comparison between the exiting results and those from this study. Despite these efforts, the conventional practice is to directly evaluate the in situ undrained shear strength (s_u) using a constant cone factor, N_{kt} , correlating the net cone tip resistance with the shear strength measured from element tests (e.g., Chan et al. 2008; Ozkul et al. 2013). Correlation with typical laboratory test (such as triaxial tests, simple shear test) data gives a range of N_{kt} from 7.2 to 18 (Low et al. 2010, 2011).

For cone penetration in layered soils, investigations were mostly undertaken for sediments with either surface or interbedded sand layer (e.g. Meyerhof and Valsangkar 1977; Van den Berg 1994; Vreugdenhil et al. 1994; Berg et al. 1996; Van den Berg et al. 1996; Lunne et al. 1997; Yue and Yin 1999; Ahmadi and Robertson 2005; Xu and Lehane 2008). In brief, Meyerhof and Valsangkar (1977), Van den Berg (1994) and Xu and Lehane (2008) performed model tests on piles and cone penetrometers in two- and three-layer soils. Van den Berg et al. (1996) presented an Eulerian analysis of the cone penetration tests in multilayer soils. Based on a finite

difference approach, analyses were carried out by Ahmadi and Robertson (2005). A linear elastic analysis studying the effects of soil layering on penetration resistance was carried out by Vreugdenhil et al. (1994). A general consensus is that the penetrometer resistance in the vicinity of the interface between the two soil layers depends on the strength and stiffness ratio between the adjacent layers and thickness of the interbedded layer relative to the cone diameter. For an interbedded strong layer with the thickness insufficient for developing the full resistance of the cone, Vreugdenhil et al. (1994) suggested correction factors as a function of the ratio of the strong layer thickness to cone diameter. Analyzing field data, chamber data, and numerical results, Robertson and Fear (1995), Youd et al. (2001), Ahmadi and Robertson (2005) and Moss et al. (2006) identified that Vreugdenhil et al.'s correction factors are way too large, and recommended a conservative band to be used for an interbedded sand layer.

For cone penetration in multilayer clays, little work has been carried out. Recently, Walker and Yu (2010) carried out analysis for two-layer stiff-soft and three-layer uniform stiff-soft-stiff clays. LDFE analyses were carried out using the commercial FE package Abaqus/Explicit. The von Mises yield criterion and its associated flow rule were assumed to model the plastic behaviour of elastoplastic undrained clays. The rigidity index of the stiff layer(s) was kept at 100 while the rigidity index of the soft layer was increased to keep the shear modulus constant as 1 MPa throughout the model. For stiff-soft-stiff clays, the thickness of the intrbedded soft layer was varied between $1.4D$ and $8.4D$. The key conclusions include: (a) cone penetration resistance in the 1st layer (of thickness $t_1 = 8.4D$) was not affected by the thickness of the 2nd layer thickness; (b) in the 2nd layer, the penetration resistance drops to a steady state penetration resistance if $t_2 > 2D$; (c) in the 1st ($I_r = 100$) layer, the profiles of stratified sediments first deviate from that of the single layer clay at a distance of $1.28D$ (from the 1st-2nd layer boundary to the cone shoulder); (d) in the 2nd ($I_r = 300$) layer, the influence of the upper layer extends up to $1.13D$ below the layer interface; (e) in the 3rd ($I_r = 100$) layer, full penetration resistance is

developed at an approximate depth of $3.43D$ beneath the 2nd-3rd layer boundary; (f) rigidity index of the 2nd layer soft clay was shown to have significant effect on these influence zones – increases with increasing I_r .

In summary, no investigation was carried out for cone penetration test in soft-stiff-soft clays. Although cone penetration tests are often carried out in layered soils, current geotechnical practice is still based on the cone factors derived from cone penetration analyses in homogeneous soils.

Objective of Present Study

This paper describes the results from large deformation FE (LDFE) analysis undertaken to provide insight into the behavior of cone penetrating through stratified soft-stiff-soft clays. The aim is to quantify the effect of an embedded stiffer layer on the cone resistance and associated deformation mechanisms. The influencing distances by the approaching soil layer as well as the trailing soil layer are analyzed to identify layer interfaces. Minimum thickness of the middle layer required to mobilize the full resistance of that layer is studied systematically. For interpreting shear strength, a correction factor is formulated where the layer thickness is insufficient to mobilize its full resistance. The effect of soil rigidity index, I_r , is also highlighted.

Numerical Analysis

Geometry and Parameters

This study has considered a cylindrical cone penetrometer of diameter D , penetrating into a three-layer deposit as illustrated schematically in Figure 1, where a stiff clay layer with undrained shear strength s_{u2} , effective unit weight γ'_2 , and thickness t_2 is sandwiched by two soft layers with identical undrained shear strength $s_{u1} = s_{u3}$ and effective unit weight $\gamma'_1 = \gamma'_3$. The thickness of the top (1st) soft layer is t_1 and that of the bottom (3rd) soft layer is (nominally)

infinite. Analyses were undertaken for the standard cone penetrometer of $D = 0.0357$ m with a 60° tip angle. The soil-cone shaft and soil-cone tip interfaces were modelled as fully smooth ($\alpha = 0$), using nodal joint elements (Herrmann, 1978). From a separate study (Ma et al. 2014), it was found that a smooth cone penetration in non-homogeneous clays resulted insignificant effect of soil strength non-homogeneity on cone penetration resistance. As such, uniform strength was considered for all three layers.

A survey was carried out through offshore geotechnical characterization reports to select realistic soil parameters for parametric study. The three-layer geometries considered here are commonly encountered in the Gulf of Thailand and Sunda Shelf, including Java Sea, as reported by Castleberry II and Prebaharan (1985); Handidjaja et al. (2004); Kostelnik et al. (2007); Chan et al. (2008); Osborne et al. (2009). For uniform three-layer clay sediments, the undrained shear strength of stiff clay ranges from 40 to 120 kPa, while that of soft clay varies between 10 and 40 kPa. The selected parameters for this study are assembled in Tables 1 and 2.

Analysis Details

Finite element analyses were performed using the finite element package AFENA (Carter and Balaam 1995) developed at the University of Sydney. H -adaptive mesh refinement cycles (Hu and Randolph 1998a) were implemented to optimize the mesh, minimizing discretization errors, concentrating in the most highly stressed zones. Large deformation analyses were undertaken using RITSS (Remeshing and Interpolation Technique with Small Strain; Hu and Randolph 1998b). This method falls within what are known as arbitrary Lagrangian-Eulerian (ALE) finite element methods (Ponthot and Belytschko 1998), whereby a series of small strain analysis increments (using AFENA) are combined with fully automatic remeshing of the entire domain, followed by interpolation of all field variables (such as stresses and material

properties) from the old mesh to the new mesh. Penetration of the cone is simulated from the seabed surface and by specifying incremental displacements. The displacement increment size and the number of steps of small strain analysis between each re-meshing were chosen such that the cumulative penetration between re-meshing stages remained in the small strain range and was less than half the minimum element size.

The axisymmetric soil domain was chosen as $100D$ in radius and $100D$ in depth to ensure that the boundaries were well outside the plastic zone. Hinge and roller conditions were applied along the base and vertical sides of the soil domain respectively. Six-noded triangular elements with three internal Gauss points were used in all the FE analyses. A typical initial mesh for the cone penetration in a three-layer soft-stiff-soft clay deposit is shown in Figure 2, with the cone tip just penetrated into the ground. A fine mesh was considered around the cone tip to ensure the accuracy of the computed results.

The soil was modelled as a linear elastic-perfectly plastic material obeying a Tresca yield criterion. The parameters needed for the model are two elastic parameters, including Young's modulus (E) and Poisson's ratio (ν). The plastic parameter used in the model is the undrained shear strength of clay (s_u), with s_u defining the size of the yield surface. The elastic parameters for clay were considered to be independent of stresses and a constant value throughout the penetration process was used. A uniform stiffness ratio of $E/s_u = 500$ was taken throughout the stratified profiles, except in the exploration of the effect of soil rigidity (I_r), where variation was set up accordingly. Considering the relatively fast penetration of the field cone penetrometer (20 mm/s), all the analyses simulated undrained conditions and adopted 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 geostatic stress conditions were modelled using $K_0 = 1$, as the stable penetration resistance

has been found to be unaffected by the value of K_0 (Zhou and Randolph 2009; Low et al. 2010). The effect of initial stress anisotropy, Δ , was not investigated, as was also ignored by Walker and Yu (2010). Although the corresponding influence on cone capacity bearing factor was reported as relatively small (Low et al. 2010), it will be explored extensively in the future.

Validation with Previous Work

For the validation exercise in single layer uniform clay as well as three-layer clay, soil-cone tip interface was modelled as fully smooth ($\alpha = 0$) and fully rough ($\alpha = 1$), and E/s_u ratio was varied with rigidity index $I_r (= G/s_u)$.

Single Layer Uniform Clay

Validations of LDFE results were conducted against existing solutions in single layer of uniform clay. Analyses were carried out varying rigidity index I_r as 50, 150, 300 and 500. Deep bearing capacity factors N_{kt} was calculated according to Equation 1. The results were compared with a number of existing solutions including those from Baligh's strain path method (Baligh 1985), the hybrid strain path method and FE analysis by Teh and Houlsby (1991), the one-step steady-state FE analysis by Yu et al. (2000), the cavity expansion solutions by Yu (2000), the LDFE analyses by van den Berg (1994), Lu et al. (2004), Walker and Yu (2006) and Liyanapathirana (2009). This comparison is presented in Figure 3. It can be seen that, for the smooth cone (Figure 3a), the cone factors derived from the current work agree well with the LDFE results by Lu et al. (2004), lower than the strain path solutions by Baligh (1985) and higher than all other results.

For the rough cone (Figure 3b), the cone factors from this study are in close agreement with the strain path solutions by Teh and Houlsby (1991), the steady state FE results by Yu et al. (2000) and the LDFE results by Lu et al. (2004). These lines are closely bracketed by other solutions although there are some outlying results, such as the values for a rough cone from

van den Berg (1994). Overall, this comparison provides confidence in the accuracy of the LDFE results from this study.

It can also be found from Figure 3 that the soil rigidity index I_r has significant effect on the bearing capacity factor N_{kt} . By comparing Figures 3a and 3b, the effect of cone roughness, α , can be quantified as 10~14%, which reduces with increasing I_r . Based on the results from this study, a correlation between N_{kt} factor and I_r and α can be proposed as

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

Two-Layer and Three-Layer Clay

Before the numerical model was used for parametric study on cone penetration in layered soils, validation of the model was carried out against the results published by Walker and Yu (2010) on stiff-soft clay and stiff-soft-stiff clay deposits (see Table 1, and also discussed under *Previous Work*). Figure 4a shows the comparison in two-layer clays with the thickness of the 1st layer $t_1 = 8.4D$ (Group SI, Table 1). Figure 4b displays the comparison in three-layer stiff-soft-stiff clays with the thickness of the 1st and 2nd layers as $t_1 = 8.4D$ and $t_1 = 2.8D$ (Group SII, Table 1). Excellent agreement can be seen between the results from this study and those reported by Walker and Yu (2010) in terms of key features of the penetration resistance: peak resistance in the 1st layer and steady-state resistance in the 2nd and 3rd layers, the depth of attaining the steady-state resistance in the 2nd and 3rd layers, and depth of turning the gradient in the 1st layer sensing the underlying soft (2nd) layer and that in the 2nd layer sensing the stiff (3rd) layer.

The good agreement with the existing data for single and stratified soils has proved that the LDFE/RITSS method can predict cone penetration behaviors in single and layered clays

accurately. The parametric study for cone penetration in stiff-soft-stiff clay deposits, covering a wide range of layer soil properties and geometry (see Table 2), is now reported.

Results and Discussion

Soil Failure Mechanisms

Soil flow mechanisms and resistance profiles during cone penetration process are directly linked. To discuss the soil flow mechanisms, a typical resistance profile is plotted in Figure 5, with $t_1/D = 16.8$, $t_2/D = 20$, $s_{u2}/s_{u1} = 4$ (in Group MI, Table 2). Six transitional stages (A_1 to A_6) are marked on the profile. Figure 6 depicts the soil flow mechanisms corresponding to these stages.

It can be seen that when the cone tip is just entering the 2nd layer at stage A_1 ($d/D = 16$; Figure 6a), the soil flow is predominantly directed laterally outward (i.e. squeezes out) being restricted within the 1st (soft) layer. With further penetration, as the cone tip penetrates into the 2nd layer (stages A_2 and A_3 , $d/D = 16.6$ and 16.9 ; Figures 6b and 6c), the proportion of soil flow in the 1st layer reduces while that in the 2nd (stiff) layer increases. The penetration resistance profile rises sharply at these stages owing to the effect of the stiff layer. The soil around the cone shoulder flows upward to the 1st layer, leading to upward deformation of the layer interface (or localized surface heave of the 2nd layer). The soil adjacent to the cone edge flows downward in the 2nd layer, mobilizing a somewhat cavity expansion type failure.

When the cone tip is fully penetrated into the 2nd layer (stage A_4 , $d/D = 29.0$; Figure 6d), the soil flow is concentrated in a limited zone beneath the cone in the 2nd (stiff) layer, leading to mobilize the peak capacity in the penetration resistance profile (see Figure 5). With the proximity of the cone to the 3rd layer, the soil flow is predominantly attracted by the underlying

soft layer, leading to a deformation of the 2nd-3rd layer interface and sharp drop in the penetration resistance profile (stage A₅, $d/D = 36.5$; see Figures 5 and 6e). Finally, when the cone penetration exceeds the 2nd-3rd layer interface by $1.2D$ ($d/D = 38.0$, Figure 6f), the cone tip becomes fully embedded in the soft soil of the 3rd layer, and the soil movement is restricted in the 3rd layer as well. It can also be seen that there is no soil from the trailing layer trapped underneath the cone tip when the cone passes both layer interfaces (Figures 6c and 6f). The cone resistance stabilizes when the soil flow is only restricted in the soil layer where it is embedded (points A₄ and A₆ in Figure 5). However, the penetration distances required to reach this stabilized resistance are different when the cone is passing an interface from soft to stiff soils or vice versa. More discussions can be found in the next section.

Identification of Layer Interface

The penetration resistance profiles are presented in terms of net bearing pressure, q_{net} , normalised by the shear strength of the 1st or 3rd layer (s_{u1} or s_{u3}), as a function of normalised penetration depth, d/D , with q_{net} calculated using Equation 1. The results for a range of the relative thickness of the 1st layer t_1/D from 4 to 20, and various strength ratios $s_{u2}/s_{u1} = 2, 4, 6$ and 8 (in Group MI, Table 2) are plotted in Figure 7.

For identifying the interface between 1st and 2nd (i.e. soft to stiff) layers, a unique distance from the kink in the curve and the interface (Δ_1) can be found. In Figure 7, all curves in the 1st layer display a similar shape where they follow the response of cone penetration in single layer and reach their steady state (if the 1st layer is thick enough with $t_1 > \sim 11D$). There is a sharp increase in the gradient of the curve when the cone senses the next stiff layer (i.e. 2nd layer). The kink (point O in Figure 7a) is defined as the intercept of the two straight lines of the curve before and after the sharp change of its gradient. From all the case studies in Figure 7, the interface between the 1st and 2nd (i.e. soft to stiff) layers is located at a distance of $0.8D$ below the kink

point 'O', regardless of the relative layer thickness t_1/D and the strength ratio of the two layers s_{u2}/s_{u1} .

For identifying the interface between the 2nd and 3rd (i.e. stiff to soft) layers, Figure 7c displays the normalized penetration resistance profiles for $t_1/D = 16.8$, $t_2/D = 15$, and strength ratios of $s_{u2}/s_{u1} = 2, 4, 6$ and 8. Although the penetration resistance profiles in 2nd (stiff) layer vary with strength ratio, the interface location can be found uniquely at $1.3D$ above the kink in the profile in the 3rd (soft) layer (referred to as Δ_2), regardless of the strength ratio and the resistance profile in the 2nd layer. In summary, layer boundaries can be identified as

$$\Delta_1 = 0.8D \quad \text{below the kink for soft - stiff layer interface} \quad (3a)$$

$$\Delta_1 = 1.3D \quad \text{above the kink for stiff - soft layer interface} \quad (3b)$$

Interpretation of Shear Strength of 1st Layer

Effect of Soil Strength, s_u

To explore the effect of the absolute value of undrained shear strength (or penetration resistance) of each layer on the form of the penetration resistance profile, the results for $s_{u1} = s_{u3} = 10$ and 30, and two strength ratios of $s_{u2}/s_{u1} = 2$ and 4 are plotted in Figure 8 for constant thickness ratios of $t_1/D = 16.8$ and $t_2/D = 15$ (in Group MI, Table 2). In the 1st and 3rd layers, the normalised penetration resistance, q_{net}/s_{u1} , profile is not influenced by the undrained shear strength and the strength ratio of s_{u2}/s_{u1} (or s_{u2}/s_{u3}) before and after the kink respectively, which are $0.8D$ and $1.3D$ above and below the interfaces (Equation 3). In the 2nd layer, absolute strength value has no influence, but the strength ratio has as normalisation is carried out by s_{u1} .

Effect of 1st Layer Thickness Ratio, t_1/D

From Figures 7 and 8, it is evident that the normalized resistance profiles in the top soft layer deviate gradually from the one for single layer uniform clay with $s_u = s_{u1}$. This deviation depicts the influence of the stiff layer, and is quantified noting the influence distance d_{inf} from the layer interface. To avoid subjectivity, d_{inf} is defined from the point where the deviation reaches 1.5%. This is also illustrated in Figure 7c. The influence point in the resistance profile appears much earlier than the kink defined previously. For quantification of d_{inf} , all values of d_{inf} from the corresponding LDFE analyses (Group MI, Table 2) are plotted in Figure 9, as a function of t_1/D . The distance increases as t_1/D increases from a value of $d_{inf} = 0.8D$ for $1 \leq t_1/D \leq 4$, and finally attains a limiting value of $10D$ for $t_1/D \geq 40$, which can be expressed as

$$\frac{d_{inf}}{D} = \begin{cases} 0.8 & \text{for } 1 \leq \frac{t_1}{D} \leq 4 \\ 4 \ln\left(\frac{t_1}{D}\right) - 4.85 & \text{for } 4 < \frac{t_1}{D} \leq 40 \\ 10 & \text{for } \frac{t_1}{D} > 40 \end{cases} \quad (4)$$

For a top soft soil layer with $t_1/D < 1$ (i.e. $t_1 < 0.0357$ m), it has no practical significance for foundation design.

Shear Strength of 1st Layer, s_{u1}

For single layer clay, the profile of bearing capacity factor N_{kt} increases with depth from a value at the soil surface ($d/D = 0$). This is associated with the mobilization of shallow general shear type soil failure mechanism. Finally at deep penetration of $d/D \geq 11$, a cavity expansion type failure is mobilized, leading to the attainment of the limiting bearing factor. This is addressed by Ma et al. (2014) and also can be seen in Figures 7 and 8. Ma et al. (2014) reported that N_{kt} factor for the smooth cone is not affected by the soil strength non-homogeneity and normalized strength $s_u/\gamma'D$, but by the soil rigidity index I_r for $d/D > \sim 2$. The limiting values of N_{kt} can be obtained using Equation 2 (e.g. $N_{kt} = 11.5$ for $I_r = 167$).

Bearing capacity factor $N_{kt} = 4.45$ at $d/D = 0$ for the smooth cone with tip angle 60° is consistent with the theoretical factor tabulated by Houlsby and Martin (2003). An expression can therefore be proposed for shallow bearing factor, with different top layer thickness relative to d_{inf} , as (see Figure 10)

$$N_{kt} = \begin{cases} 4.45 - 0.114 \left(\frac{d}{D} \right)^{1.5} + 3.31 \left(\frac{d}{D} \right)^{0.5} & \text{for } \frac{t_1 - d_{inf}}{D} < 11 \\ 11.5 & \text{for } \frac{t_1 - d_{inf}}{D} \geq 11 \end{cases} \quad (5)$$

Equation 5 can be used in practice for interpreting shear strength for the 1st layer.

Interpretation of Shear Strength of 2nd Layer

Effect of 2nd Layer Thickness Ratio, t_2/D

For soft-stiff-soft clay deposits, the thickness of the interbedded stiff layer may not be sufficient for mobilizing the full resistance of that layer prior to its reduction due to the effect of the underlying soft layer. This is evident in Figure 7c, where a thickness of $t_2/D = 15$ only allows the full resistance of the layer to be mobilized for s_{u2}/s_{u3} ($= s_{u2}/s_{u1}$, but physically more relevant) $= 2$. For $s_{u2}/s_{u1} \geq 4$, the peak resistance is smaller than the corresponding full resistance of the layer. Accordingly in the field applications, the shear strengths of these thin stiff layers are often inaccurately interpreted and treated as relatively softer layers. This is also addressed by Ahmadi and Robertson (2005) for a thin sand layer sandwiched by two uniform clay layers, with a correction factor proposed to be applied to the measured cone resistance. A similar approach is employed here as discussed below.

The analyses included in Group MII (Table 2) explore the effect of t_2/D on peak penetration resistance in the 2nd layer. Normalized penetration resistance profiles are plotted in Figure 11 for a range of t_2/D from 2 to 20, but with identical $t_1/D = 16.8$ and $s_{u2}/s_{u1} = s_{u2}/s_{u3} = 4$. To

quantify the reduction in normalized resistance (or N_{kt}), two vertical lines corresponding to the response on uniform clays with strength $s_{u1} = s_{u3} = 10$ kPa and $s_{u2} = 40$ kPa are also included in the figure. Following Robertson and Fear's (1995) suggestion, the correction factor k is defined as the ratio of the actual full net resistance $q_{net2,f}$, to measured peak net resistance $q_{net2,m}$

$$k = \frac{q_{net2,f}}{q_{net2,m}} \quad (6)$$

The calculated value of k for each curve is given on Figure 11. For $t_2/D = 2$, the resistance correction factor is 1.64, which reduces to 1.03 for $t_2/D = 15$.

Effect of Strength Ratio, s_{u2}/s_{u3}

To quantify the effect of strength ratio s_{u2}/s_{u3} ($= s_{u2}/s_{u1}$), another group of analyses were conducted with a thinner middle layer ($t_1/D = 16.8$ and $t_2/D = 5$) and the strength ratios of 2, 4, 6 and 8 (Group MIII, Table 2). The results are plotted in Figure 12. The vertical dotted lines correspond to the full/limiting resistance of the middle layer. The values of correction factor k are labelled in Figure 12 - increases with increasing strength ratio. This is because, for an object penetrating from a stiff layer to a soft layer (i.e. 2nd to 3rd layer), as strength ratio s_{u2}/s_{u3} increases, the soil deformation is attracted by the underlying soft clay layer earlier. The correction factor k is 1.13 for $s_{u2}/s_{u3} = 2$, and increases to 1.33 for $s_{u2}/s_{u3} = 8$.

Shear Strength of 2nd Layer, s_{u2}

To propose a general expression for the resistance correction factor k , more LDFE analyses were carried out in Group MIV (Table 2). A combination of normalized soil properties and layer geometries were considered: $t_1/D = 16.8$, $t_2/D = 2 \sim 20$, $s_{u2}/s_{u3} = 2 \sim 8$. The corresponding values of k (for $I_{r1} = I_{r2} = I_{r3} = 167$) are listed in Table 3 and presented graphically in Figure 13. It is apparent that the correction factor diminishes as t_2/D increases and becomes negligible for

370 $t_2/D = 20$. A best fit through the data allows the correction factor to be approximated as (see
 371 Figure 13)

$$372 \quad k = \begin{cases} 0.68 \frac{(s_{u2}/s_{u3})^{0.5}}{(t_2/D)} + 0.93 & \text{for } \frac{(s_{u2}/s_{u3})^{0.5}}{(t_2/D)} > 0.1 \\ 1 & \text{for } \frac{(s_{u2}/s_{u3})^{0.5}}{(t_2/D)} \leq 0.1 \end{cases}$$

373 (7)

374 To deduce the full resistance of the middle layer with the aim of accurate interpretation of its
 375 strength, Equation 6 can be rearranged as

$$376 \quad q_{\text{net2,f}} = k \cdot q_{\text{net2,m}} \quad (8)$$

377 **Effect of Soil Rigidity Index**

378 In the above analyses on soft-stiff-soft deposits, the rigidity index of all three layers has been
 379 considered as equal $I_{r1} = I_{r2} = I_{r3} = 167$. From the validation exercise and the exploration on
 380 single layer clays (Ma et al. 2014) and stiff-soft-stiff clays (Walker and Yu 2010), it was found
 381 that the soil rigidity index I_r has a remarkable influence on the cone penetration resistance (or
 382 bearing factor N_{kt}) that increases with increasing I_r (Equation 2). To explore this influence
 383 comprehensively, analyses have been undertaken for soft-stiff-soft deposit with $t_1/D = 16.8$ and
 384 $I_{r1} = I_{r3} = 167$, but varying I_{r2} between 167 and 333 for $t_2/D = 15$ and $s_{u2}/s_{u3} = 2$ to 8 (see Figure
 385 14; Group MV, Table 2). It can be seen that, regardless of strength ratio, $I_{r2} = 333$ has resulted
 386 an about 10% difference in normalized resistance of the middle layer. Figure 14 also indicates
 387 that the correction factor increases with increasing I_{r2} . For instance ($s_{u2}/s_{u3} = 6$), $k = 1.03$ for I_{r2}
 388 $= 167$, which increases to 1.06 for $I_{r2} = 333$. An extensive investigation on quantifying the
 389 effect of soil rigidity index for CPT in soft-stiff-soft clay will be presented in a forthcoming
 390 paper.

Procedures for Characterising Soft-Stiff-Soft Clay Deposit

Based on the formulas proposed and discussed previously, a design chart is established in Figure 15 to interpret soil layer boundaries and the undrained shear strength of each identified layer. After the identification of layer interfaces (and hence t_1/D and t_2/D), the strength of the top layer s_{u1} can be obtained relatively directly. However, since the interpretation of the strength of the middle layer s_{u2} is a function of strength ratio s_{u2}/s_{u3} , an iteration is necessary until the suggested (i.e. initial) and calculated (i.e. updated) strengths matches with the difference $< 3\%$.

Concluding Remarks

This paper reports the results from LDFE analysis using the RITSS method, simulating continuous penetration of the standard cone penetrometer from the seabed surface. The extensive investigation on single layer and soft-stiff-soft clay deposits encompassed a range of normalised soil properties, layer geometries and roughnesses of the cone tip. A new design framework was proposed for interpreting the layer boundaries and undrained shear strength of each identified layer (Figure 15). The soft-stiff and stiff-soft layer boundaries can respectively be demarked at a distance of $0.8D$ below and $1.3D$ above the kink in the penetration resistance profile of the soft layers (Equation 3). The undrained shear strength of the top and bottom layers can be interpreted directly, whereas an iteration is required for calculating the shear strength of the middle layer. The effect of soil rigidity index was highlighted, which will specifically be focused in a forthcoming paper.

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419

420 **Notation**

421	α	cone-soil interface friction ratio
422	D	diameter of cone penetrometer
423	d	penetration depth of cone shoulder
424	d_{inf}	distance affected by 2 nd stiff layer
425	E	Young's modulus
426	G	shear modulus
427	I_r	rigidity index
428	k	resistance correction factor in 2 nd layer
429	N_{kt}	bearing capacity factor of cone penetrometer
430	q_c	measured cone resistance
431	q_{net}	net cone resistance
432	$q_{net2,f}$	full net resistance of 2 nd stiff layer
433	$q_{net2,m}$	measured net resistance of 2 nd stiff 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	t_1, t_2, t_3	thickness of 1 st , 2 nd , 3 rd layer
437	γ'	soil effective unit weight
438	σ_{v0}	in situ total overburden stress
439	Δ	initial stress anisotropy = $(\sigma_{v0} - \sigma_{h0})/2s_u$
440	Δ_1	distance from kink (sharp rise point) to 1 st -2 nd layer interface
441	Δ_2	distance from 2 nd -3 rd layer interface to merging point

442

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Table 1 Validation against results of Walker and Yu (2010) for stiff-soft and stiff-soft-stiff clays

Analyses	Layer 1			Layer 2			Layer 3			D (m)
	t_1/D	I_{r1}	s_{u1} (kPa)	t_2/D	I_{r2}	s_{u2} (kPa)	t_3/D	I_{r3}	s_{u3} (kPa)	
Group SI	8.4	10 0	10	∞	300	3.3	-	-	-	0.035 7
Group SII	8.4	10 0	10	2.8	300, 500	3.3	∞	10 0	10	

For all the cases: $\gamma'_1 = \gamma'_2 = \gamma'_3 = 6 \text{ kN/m}^3$; $\alpha = 0$ (for both shaft and tip); $G = 1 \text{ MPa}$; penetrating from soil surface

Table 2 Summary of LDFE analyses performed on soft-stiff-soft clays

Analyses	layer 1		layer 2			layer 3		Remarks
	t_1/D	s_{u1} (kPa)	t_2/D	s_{u2}/s_{u1} = s_{u2}/s_{u3}	I_{r2}	t_3/D	s_{u3} (kPa)	
Group MI	4, 10, 16.8, 20	10, 30	15, 20	2, 4, 6, 8	167	∞	10, 30	Identify layer interface and d_{inf}
Group MII	16.8	10	2, 5, 10, 15, 20	4	167	∞	10	Effect of t_2/D on peak penetration resistance in 2 nd layer
Group MIII	16.8	10	5	2, 4, 6, 8	167	∞	10	Effect of s_{u2}/s_{u3} on peak penetration resistance in 2 nd layer
Group MIV	16.8	10	2, 5, 10, 15, 20	2, 4, 6, 8	167	∞	10	Quantify k

Group MV	16.8	10	5, 15	2, 4, 6, 8	167, 333	∞	10	Effect of I_r
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For all the cases: $\gamma'_1 = \gamma'_2 = \gamma'_3 = 6 \text{ kN/m}^3$; $\alpha = 0$ (for both shaft and tip); $I_{r1} = I_{r3} = 167$; $D = 0.0357 \text{ m}$; penetrating from soil surface

Table 3 Summary of correction factor k ($I_{r1} = I_{r2} = I_{r3} = 167$)

t_2/D	s_{u2}/s_{u3}	k
2	2	1.33
	4	1.64
	6	1.77
	8	1.85
5	2	1.14
	4	1.25
	6	1.30
	8	1.32
10	2	1.04
	4	1.06
	6	1.08
	8	1.09
15	2	1.01
	4	1.02
	6	1.03
	8	1.04
20	2	1.00
	4	1.00
	6	1.00
	8	1.01

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- Figure 1.** Schematic diagram of cone penetration in soft-stiff-soft clay
- Figure 2.** Initial mesh for cone penetration in soft-stiff-soft clay (axes normalized by cone diameter D)
- Figure 3.** Comparison of cone factor N_{kt} with existing solutions in uniform clay: (a) Smooth ($\alpha = 0$); (b) Rough ($\alpha = 1$)
- Figure 4.** Comparison with existing numerical results in stratified clays: (a) $G = 1$ MPa, stiff-soft clay ($t_1/D = 8.4$; Group SI, Table 1); (b) $G = 1$ MPa, stiff-soft-stiff clay ($t_1/D = 8.4$, $t_2/D = 2.8$; Group SII, Table 1)
- Figure 5.** Typical penetration resistance profile in soft-stiff-soft clay ($t_1/D = 16.8$, $t_2/D = 20$, $s_{u2}/s_{u1} = 4$; Group MI, Table 2)
- Figure 6.** Soil flow mechanisms at different penetration stages in soft-stiff-soft clay ($t_1/D = 16.8$, $t_2/D = 20$, $s_{u2}/s_{u1} = 4$; Group MI, Table 2)
- Figure 7.** Identification of layer interfaces (Group MI, Table 2): (a) 1st-2nd layer interface: effect of t_1/D ($s_{u2}/s_{u1} = 4$, $t_1/D = 4 \sim 20$); (b) 1st-2nd layer interface: effect of s_{u2}/s_{u1} ($s_{u2}/s_{u1} = 2 \sim 8$, $t_1/D = 4 \sim 20$); (c) 2nd-3rd layer interface ($t_1/D = 16.8$, $t_2/D = 15$)
- Figure 8.** Effect of absolute strength of 1st and 2nd layers ($t_1/D = 16.8$, $t_2/D = 15$; Group MI, Table 2)
- Figure 9.** Design chart for distance (d_{inf}) influenced by middle stiff layer ($t_1/D = 2 \sim 20$, $t_2/D = 10$; Group MI, Table 2)

- Figure 10.** Design chart to calculate bearing factor N_{kt} at both shallow ($t - d_{inf} < 11D$) and deep ($t - d_{inf} \geq 11D$) penetration depths ($I_{r1} = I_{r2} = I_{r3} = 167$; Group MI, Table 2)
- Figure 11.** Effect of 2nd layer thickness ratio t/D on normalized penetration resistance ($t/D = 16.8$, $s_{u2}/s_{u1} = 4$; Group MII, Table 2)
- Figure 12.** Effect of strength ratio s_{u2}/s_{u3} on normalized penetration resistance ($t/D = 16.8$, $t/D = 5$, $s_{u1} = 10$ kPa; Group MIII, Table 2)
- Figure 13.** Design chart for correction factor k ($I_{r1} = I_{r2} = I_{r3} = 167$)
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- Figure 15.** Procedure for interpretation of layer boundaries and undrained shear strength from measured CPT data

Figure 1. Schematic diagram of cone penetration in soft-stiff-soft clay

Figure 2. Initial mesh for cone penetration in soft-stiff-soft clay (axes normalized by cone diameter D)

3(a) Smooth ($\alpha = 0$)

3(b) Rough ($\alpha = 1$)

Figure 3. Comparison of cone factor N_{kt} with existing solutions in uniform clay

4(a) $G = 1$ MPa, stiff-soft clay ($t_1/D = 8.4$; Group SI, Table 1)

4(b) $G = 1$ MPa, stiff-soft-stiff clay ($t_1/D = 8.4$, $t_2/D = 2.8$; Group SII, Table 1)

Figure 4. Comparison with existing numerical results in stratified clays

Figure 5. Typical penetration resistance profile in soft-stiff-soft clay ($t_1/D = 16.8$, $t_2/D = 20$, $s_{u2}/s_{u1} = 4$; Group MI, Table 2)

Figure 6. Soil flow mechanisms at different penetration stages in soft-stiff-soft clay ($t_1/D = 16.8$, $t_2/D = 20$, $s_{u2}/s_{u1} = 4$; Group MI, Table 2)

7(a) 1st-2nd layer interface: effect of t_1/D ($s_{u2}/s_{u1} = 4$, $t_1/D = 4 \sim 20$)

7(b) 1st-2nd layer interface: effect of s_{u2}/s_{u1} ($s_{u2}/s_{u1} = 2 \sim 8$, $t_1/D = 4 \sim 20$)

7(c) 2nd-3rd layer interface ($t_1/D = 16.8$, $t_2/D = 15$)

Figure 7. Identification of layer interfaces (Group MI, Table 2)

Figure 8. Effect of absolute strength of 1st and 2nd layers ($h/D = 16.8$, $h/D = 15$;
 Group MI, Table 2)

Figure 9. Design chart for distance (d_{inf}) influenced by middle stiff layer ($t_1/D = 2 \sim 20$, $t_2/D = 10$; Group MI, Table 2)

Figure 10. Design chart to calculate bearing factor N_{kt} at both shallow ($t - d_{inf} < 11D$) and deep ($t - d_{inf} \geq 11D$) penetration depths ($I_{r1} = I_{r2} = I_{r3} = 167$; Group MI, Table 2)

Figure 11. Effect of 2nd layer thickness ratio t_2/D on normalized penetration resistance

($t_1/D = 16.8$, $s_{u2}/s_{u1} = 4$; Group MII, Table 2)

Figure 12. Effect of strength ratio s_{u2}/s_{u3} on normalized penetration resistance ($t_1/D = 16.8$, $t_2/D = 5$, $s_{u1} = 10$ kPa; Group MIII, Table 2)

Figure 13. Design chart for correction factor k ($I_{r1} = I_{r2} = I_{r3} = 167$)

Figure 14. Effect of soil rigidity index I_{r2} on normalized penetration resistance

($t_2/D = 15$, $s_{u2}/s_{u1} = 2 \sim 8$; Group MIV, Table 2)

Figure 15. Procedure for interpretation of layer boundaries and undrained shear strength from measured CPT data