State Parameter Estimation by CPTu Interpretation for Liquefaction Susceptibility - A Comparison of Methods

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ABSTRACT
The attention to liquefaction susceptibility of mine tailings has recently increased following widely publicized tailings dam failures including the Fundao tailings dam failure in Brazil. Cone penetration testing with pore pressure measurements is widely used within the geotechnical industry as a means to estimate in-situ properties of tailings materials, including state parameter. These properties are a key part of assessing the liquefaction potential of a tailings deposit. There are several methods for assessing state parameter based on CPT and laboratory data. This paper reviews three of these methods and applies them to CPT data from a case study to compare these different methods, and their results.

RÉSUMÉ
L'attention portée au potentiel de liquéfaction d'un sol s'est récemment accrue à la suite de ruptures de barrage largement médiatisées dans les mines du monde entier, y compris la rupture du barrage de résidus miniers de Fundao au Brésil. Dans le but d'évaluer le potentiel de liquéfaction d'un dépôt de résidus miniers, l'un des moyens est d'estimer les propriétés in situ de ces matériaux, tel que le paramètre d'état, par l'essai de pénétration au cône (CPT) avec mesure de la pression d'eau interstitielle. Différentes approches peuvent être utilisées dans le but d'évaluer le paramètre d'état a l'aide de la base de données CPT et de laboratoire. Cet article passe en revue une sélection de ces méthodes et les applique aux données CPT d'une étude de cas. Les méthodes sélectionnées et leurs résultats sont comparés et discutés.

1 INTRODUCTION

Cone penetration testing with pore pressure measurements (CPTu) is widely used within the geotechnical industry as a means to characterize in-situ properties of tailings materials such as void ratio, effective stress, state parameter and residual shear strength (Jefferies and Been 2016). Methods for estimating the in-situ state of tailings from CPTu data include Plewes et al. (1992), Robertson (2010), and Jefferies and Been (2016). However, the degree of laboratory testing on physical samples and analysis effort required can vary considerably between these methods. In this paper, the background, applicability and limitations of these methods for calculating in-situ state parameter for mine tailings are discussed. The methods are then each applied to calculate the in-situ state parameter using case study data from the Fundão tailings storage facility.

2 BACKGROUND

Liquefaction is the sudden loss of shear strength due to strain softening (Robertson 2010). Liquefaction in the foundation of tailings dams can result in sudden failure of those structures, with minimal indicators or warnings in advance of the failure. The susceptibility to liquefaction of a material can be estimated through application of critical state soil mechanics. A brief description of the method as it is currently practiced (Jefferies and Been 2016, Robertson 2010b) is provided below.

Depending on its density, a soil matrix may exhibit either contractive or dilative behaviour when it is sheared. A saturated, dense soil matrix, tries to expand when sheared, but in an undrained condition (i.e., no change in total volume), generates negative pore pressures which increase the effective and interparticle stresses, and results in a strength gain during shearing. Alternatively, when a saturated, loose soil matrix is sheared, it will contract, but in an undrained condition, positive pore pressures are generated which reduce the effective interparticle stresses, resulting in strength loss, which in some cases can also be very brittle. The term "flow liquefaction" arises from the condition with sloping ground where the static shear stress greatly exceeds not only the brittle liquefied residual strength, but also the peak undrained strength, and a sudden accelerating shear failure takes place.

When sheared to large strains, a soil will reach a state where its volume reaches a constant value and its rate of volume change becomes zero. This is referred to as the critical state. Dense soils will dilate to the critical state and loose soils will contract to the critical state. The boundary line between dilative and contractive behaviour can be determined by laboratory testing and plotted on a graph of void ratio versus mean effective stress. It is referred to as the critical state locus (CSL). The difference between the in-situ void ratio (e) and the critical state void ratio (e_c) is the state parameter, Ψ, as defined in Figure 1 (Jefferies and Been 2016). Soils with a positive Ψ (i.e., in-situ void ratio greater than the critical state void ratio) are contractive and soils with a negative Ψ (i.e., in-situ void ratio less than the critical state void ratio) are dilative. However, a state parameter Ψ equal to -0.05 has been commonly adopted as the boundary between contractive and dilative behaviour (Robertson 2010a, Jefferies and Been 2016).

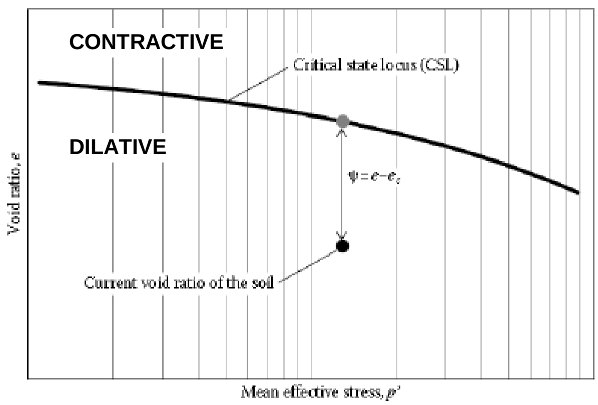

Figure 1. Definition of state parameter (Jefferies and Been 2016)

q_t = CPT corrected cone resistance
f = CPT unit sleeve friction
u_2 = CPT dynamic pore water pressure, measured behind the cone tip
u_0 = Equilibrium pore water pressure
σ_{v0} = In-situ total vertical stress
σ'_{v0} = In-situ total effective vertical stress
p_0 = In-situ mean total stress
p'_0 = In-situ mean effective stress
p_a = Atmospheric pressure
G = Shear modulus

The next section discussed three methods commonly used to characterize the in-situ properties and liquefaction potential of mine tailings based on analysis of CPT data. The following variables are referenced frequency and are introduced below for simplicity:

3 METHODS FOR ESTIMATING STATE PARAMETER

The following sections discuss three methods for estimating in-situ state parameter commonly used in geotechnical practice: Plewes et al. (1992), Robertson (2010a) and Jefferies and Been (2016).

The Plewes et al. (1992) and Robertson (2010a) methods are screening-level assessments; they are quick and relatively easy to perform as they require little to no laboratory testing and no numerical analyses. These methods may be appropriate for low risk projects or a ‘first look’ at higher risk projects. The third method is a ‘detailed’ method advanced by Jefferies and Been (2016) that involves index and advanced laboratory testing and numerical cavity expansion simulations. This method can be time consuming and expensive but may be appropriate for high risk projects.

3.1 Plewes et al. (1992)

Plewes et al. (1992) presented a simple screening procedure to estimate state parameter from CPT data. The procedure uses q_t, f, and pore pressure measurements. It does not require index or advanced laboratory testing. This method is commonly employed as an extremely useful ‘first look’ at CPT data in practice (Jefferies and Been 2016).

This method evolved from an early method for estimating state parameter from CPT data proposed by Been et al. (1987) that was based on correlation with re-processed CPT chamber calibration test results for fine to medium clean sands. Been et al. (1986, 1987) showed that the relationship between stress normalised CPT tip resistance and state parameter could be described by Equation 1 for drained pore pressure conditions. The CPT resistance normalised by mean effective stress (Q_p) is shown in Equation 2, and k and m are soil specific constants.

$$Q_p = k \exp(-m\psi) \quad [1]$$

$$Q_p = (q_t - p_0)/p'_0 \quad [2]$$

In Plewes et al. (1992), CPT penetration resistance is normalised based on mean effective stress and dynamic pore water pressure by the grouping $Q_p(1-B_q)+1$. The addition of the $(1-B_q)+1$ term accounts for the effect of drainage conditions. A theoretical basis for this approach to undrained conditions was provided later by Shuttle and Cunning (2007).

The relationship between $Q_p(1-B_q)+1$ and ψ is given by Equation 3. The pore pressure ratio (B_q) is shown in Equation 4. Soil specific constants $\bar{k}$ and $\bar{m}$ are semi empirical correlations of the slope of the critical state line λ_{10} and the critical state friction ratio M_{tc} (Equations 5 and 6). Note that caution is needed when looking at quoted values of λ as both log base 10 and natural logarithms are used. In this paper the notation λ_{10} and λ_e are used respectively.

$$Q_p(1 - B_q) + 1 = \bar{k} \exp(-\bar{m}\psi) \quad [3]$$

$$B_q = (u - u_0)/(q_t - \sigma'_{v0}) \quad [4]$$

$$\bar{k} / M_{tc} = 3 + 0.85/\lambda_{10} \quad [5]$$

$$\bar{m} = 11.9 - 13.3\lambda_{10} \quad [6]$$

Plewes et al. (1992) indicated λ_{10} could be estimated based on empirical correlation with friction ratio (F) (Equation 8) and a value of M_{tc} = 1.2 could be used for all soils. The Plewes et al. (1992) method can be refined by conducting laboratory triaxial testing on intact or reconstituted soil specimens to develop the soil CSL and thereby refine estimates of M_{tc} and λ_{10}. Experience has shown that the typical range of M_{tc} for tailings is 1.3 to 1.6 (Reid 2015, Jefferies and Been 2016). Therefore, in practice, if a refined estimate of M_{tc} based on laboratory testing is unavailable, M_{tc} = 1.4 is typically used in screening analyses. In this paper, M_{tc} = 1.2 is used as recommended by Plewes et al. (1992) in one analysis and the results are compared to a second analysis where M_{tc} and λ_{10} were refined based on laboratory testing to illustrate the importance of picking reasonable inputs in screening analyses.

$$F = f*100/(q_t - \sigma_{v0}\% \quad [7]$$

$$\lambda_{10} = F / 10 \quad [8]$$

3.2 Robertson (2010a)

Robertson (2010a) is a simple screening procedure to estimate state parameter from CPT data. It is an extension of the empirical CPT interpretation framework presented by Robertson (2009, 2010b) that allows results to be interpreted within a critical state soil mechanics framework. The procedure uses q_t and f but not pore pressure measurements. It does not require index or advanced laboratory testing. Like Plewes et al. (1992), this method is commonly employed as an extremely useful 'first look' at CPT data in practice.

In the Robertson (2009, 2010b) framework, CPT penetration resistance is normalised by mean effective stress and an empirical indicator of soil properties known as the Soil Behaviour Type Index (I_c) as shown in Equations 9, 10 and 11. In Equation 9, $(q_t - \sigma_v)/p_a$ is the dimensionless net CPT resistance, $(p_a/\sigma'_v)^n$ is a stress normalisation factor and n is a stress exponent that various with soil behaviour index (i.e., by I_c).

$$Q_{tn} = [(q_t - \sigma_v)/p_a]\ (p_a/\sigma'_v)^n \quad [9]$$

$$n = 0.381\ I_c + 0.05\ (\sigma'_v / p_a) - 0.15,\ n \leq 1 \quad [10]$$

$$I_c = [3.47 - \log_{10}Q_t)^2 + (\log_{10}F + 1.22)^2]^{0.5} \quad [11]$$

Normalised CPT penetration resistance is then adjusted to account for the effect of fines content, mineralogy and plasticity using a correction factor, K_c. The K_c factor was developed by Robertson and Wride (1998) based on a database of case histories and is used to normalise cone resistance in silty sands (Q_{tn}) to an equivalent clean sand value ($Q_{tn,cs}$) as shown in Equations 12, 13 and 14.

$$Q_{tn,cs} = K_c\ Q_{tn} \quad [12]$$

$$\text{If } I_c \leq 1.64,\ K_c = 1.0 \quad [13]$$

$$\text{If } I_c > 1.64,\ K_c = 5.581I_c^3 - 0.403I_c^4 - 21.63I_c^2 + 33.75I_c - 17.88 \quad [14]$$

Robertson (2009) developed contours of $Q_{tn,cs}$ and ψ on the Q_{tn} versus F dimensionless CPT classification plot based on analysis of case histories. The two sets of contours were very similar and, on this basis, Robertson (2010a) developed a simplified and approximate empirical relationship between $Q_{tn,cs}$ and ψ for sandy soils (i.e., I_c < 2.6) as shown in Equation 15.

$$\psi = 0.56 - 0.33\ \log_{10}Q_{tn,cs} \quad [15]$$

3.3 Jefferies and Been (2016)

Jefferies and Been (2016) is a detailed mechanics based procedure to calculate state parameter from CPT data. The procedure uses q_t, f and pore pressure measurements. It requires collection of physical samples for index and advanced laboratory testing as well as in-situ testing to measure shear modulus and geostatic stress ratio (K_0). This method is used more often for high risk projects.

In the Jefferies and Been (2016) method soil behaviour in CPT penetration is based on numerical analysis using a generalized critical state soil model (e.g., NorSand). This method involves in-situ shear wave velocity testing and laboratory triaxial testing on reconstituted specimens. Results are used to calibrate a soil-specific critical state constitutive model (e.g., NorSand). Numerical cavity expansion simulations are then completed using the calibrated constitutive model to develop soil-specific CPT calibration constants (k, m, $\bar{k}$, $\bar{m}$) for the relationship between CPT penetration resistance and state parameter for drained and undrained conditions.

The Been et al. (1987a, b) method (Equation 1) is used for drained conditions and the Shuttle and Cunning (2007) method (Equation 3) is for undrained conditions.

4 APPLICATION TO FUNDÃO TAILINGS STORAGE FACILITY CASE STUDY

4.1 Case Study Background

An example of the importance of the estimation of state parameter of soils for the purpose of liquefaction susceptibility assessment, is the Fundao Tailings Dam in MInas Gerais, Brazil. In November 2015, a flow liquefaction started on the left abutment of the Fundao Tailings Dam. The failure developed quickly and the resulting loss of containment resulted in multiple fatalities and was the largest environmental disaster in Brazilian history (Morgenstern et al. 2016, Fonseca and Fonseca 2016). This was a widely publicized failure and has been a topic of discussion and research in the geotechnical community since its occurrence.

There are other case studies that could have been appropriate for this paper, but Fundao was selected for several reasons; 1) this is a liquefaction case study which many industry practitioners are familiar with, 2) there is a publicly available report which includes site investigation data, and 3) there is both CPT and lab testing that was completed on the sand tailings. While the raw site investigation and laboratory data was not accessible, the report did contain enough information to perform the various state parameter estimates described in Section 3.

CPTu data was available for six site investigation programs at the Fundao Tailings Dam and surrounding structures. For this paper, the authors chose to focus on four test holes in particular; GCCPT16-03, GCCPT16-04, GCCPT16-04B, GBCPT16-06. While these specific CPTs are not on the dam that failed in 2015, there are reasons why they were chosen. These tests were done on the Germano Pit Dam and the Germano Buttress. In these areas, there were no reasons to expect interbedded tailings slimes (Morgenstern et al 2016). This meant that all of the data from these traces could be used to estimate the state parameter of the sand tailings based on the above-mentioned methods, without filtering out the slimes layers. These CPTs were also part of the site investigation program carried out as part of the investigation into the cause of the failure, which improved the authors confidence in the attention to detail and an emphasis on quality data.

4.2 Cavity Expansion Simulations

In the Jefferies and Been (2016) method, numerical modelling is completed to determine the relationship between ψ and the CPT resistance. This involves numerically simulating spherical cavity expansion induced by the CPT with a soil-specific constitutive model (e.g. NorSand). The results of the simulations are used to estimate soil-specific CPT calibration constants.

The finite element cavity expansion program, referred to as "CPTwidget", that was used for the CPT calibration in this work was the most recent version of the program (Version 3.1) (Shuttle 2018). The "CPTwidget" runs a series of finite element simulations using the NorSand constitutive model with drained or undrained boundary conditions. Key inputs to the cavity expansion simulations are the elasticity and plasticity parameters assessed from soil-specific NorSand model calibration as well as mean effective stress (p'), Ψ, and rigidity index (I_r = G/p_0) for which the simulations are to be completed. The key output of the cavity expansion simulations are Q_p and $Q_p(1\text{-}B_q)$ +1 for drained and undrained conditions respectively. The relationship between Q_p or $Q_p(1\text{-}B_q)$ +1 and Ψ can be interpreted from the trend of the output data.

The input NorSand soil properties that were used are summarized in Table 1. These properties were developed using laboratory data and numerical analysis and presented by Morgenstern et al. (2016). Simulations were run with both drained and undrained boundary conditions, however CPT penetration through the sand tailings considered in this paper was typically drained ($-0.02 < B_q < 0.02$). The ψ used in the simulations was varied between -0.2 and 0.2. The I_r and p' were varied to assess the sensitivity of results to these parameters. The drained (k, m) and undrained (k', m') coefficients were determined by curve fitting Equation 1 and 3 to the simulation results. Example of this process are shown in Figures 2 and 3 for the simulation results from the CPT calibration with drained boundary conditions for p´ = 100 kPa and p´ = 1250 kPa respectively. The calculated drained and undrained CPT calibration constants are summarized in Table 1. The cavity expansion simulations were relatively insensitive to p' but showed some sensitivity to I_r at lower state parameters. Over the range of p' to for the CPT data from the Germano Pit Dam and Germano Buttress, I_r ranges from approximately 75 to 250.

4.3 Data Processing

The CPTu traces from the Germano Pit Dam and the Germano Buttress were presented in PDF form in Appendix C7 of the Panel Investigation Report (Morgenstern et al. 2016). PDFs to were digitized using an open source web-based plot digitizer. Data points were extracted to simulate a sampling interval of 0.05 m. A shorter sampling interval could have been used at this step to give more data points, but the step size chosen gave an adequate number of data points for the analyses completed for this paper.

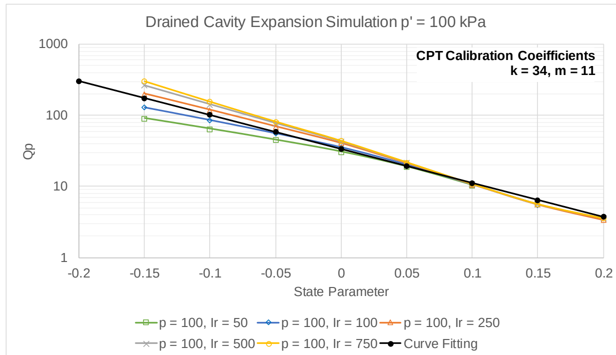

Figure 2. Cavity expansion simulations results with drained boundary condition for p' = 100 kPa.

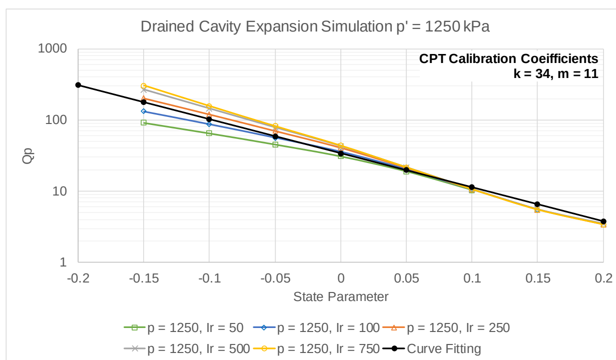

Figure 3. Cavity expansion simulations results with drained boundary condition for p' = 1250 kPa.

Once the raw data had been extracted, it was processed in a VBA excel sheet (Jefferies and Been 2018) to normalise the CPT parameters and calculate the state parameters by the Plewes method, the Plewes method with lab determined CSL values, and the Jefferies and Been method. In these analyses, a K_0 value of 0.5 was used based on Morgenstern et al. (2016). In addition to these three methods, the Robertson 2010 state parameters were estimated using the normalised CPT parameters and soil behavior information from the VBA program. The soil parameters used for calculations are shown in Table 1.

Table 1. Data Processing Parameters for Tailings Sands

Property	Description	Sand Tailings
Γ	Reference void ratio on CSL, conventionally defined at p' = 1 kPa	0.865
λ_e	Slope of CSL, for semi-log Idealisation (base e)	0.024
M_{tc}	Critical friction ratio	1.33
N_{tc}	Volumetric coupling parameter	0.38
χ_{tc}	State dilatancy constant	7.3
H	Plastic hardening modulus	156-756*ψ
$\bar{k}$	Undrained analysis coefficient	45
$\bar{m}'$	Undrained analysis exponent	7
k	Drained analysis coefficient	34
m	Drained analysis exponent	11

Once the state parameters were calculated they were considered in two ways. First was to look at the distribution of the state parameters for the four CPT holes based on the different methods. Looking at the distribution of the data is important as this can help identify if there is a meaningful trend in state parameter for site investigation data that is being classified together as one soil. It is also important as certain methods (Jefferies and Been 2016) require the use of a certain percentile as the "characteristic state" that is representative of the soil mass. Jefferies and Been (2016) recommend using the 80th percentile state parameter for static design problems, and the 90th percentile for dynamic problems. The logic for this recommendation is described in Chapter 5 of Jefferies and Been (2016) and is not discussed further. In contrast, Robertson (2010b) recommends the use of the mean value for design. Figure 4 shows the state parameter distributions for each calculation method. Table 2 shows the detailed statistical information for each of the state parameter calculation methods.

In addition to comparing the statistical distribution of all calculated state parameters, the state parameters with depth, and the differences between methods for three of the CPTs (GCCPT16-03, GCCPT16-04, and GCCPT16-04b) were analyzed further. The results of this are shown in Figure 8. For comparison, the characteristic state given by Morgenstern et al. (2016) was 0.012. However, the method and CPTs used for this estimation were not specified. It is likely that the CPTs directly adjacent to the Fundao Tailings Dam, while the values in Table 2 are from the Germano pit Dam and the German Buttress as discussed in Section 4.1.

4.4 State Parameter Discussion

The author's intent at the outset of this paper was to show the sensitivity of calculation of state parameter by various methods for a given set of data. The test data processed and shown in Figures 4 and 5 did reveal four main trends that are worth noting.

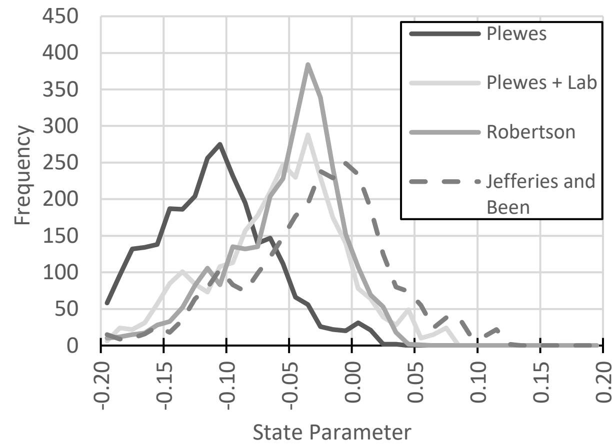

Figure 4. Distribution of state parameters calculated by the various methods

Table 2. Detailed distribution values for all state parameters calculated by various calculation methods

	Plewes	Plewes with lab	Robertson	Jefferies and Been
10th percentile	-0.190	-0.143	-0.119	-0.117
20th percentile	-0.162	-0.109	-0.089	-0.082
Mean	-0.120	-0.063	-0.054	-0.033
Median	-0.110	-0.050	-0.040	-0.020
80th percentile	-0.072	-0.014	-0.014	0.018
90th percentile	-0.050	0.005	0.001	0.042

First, the distribution of state parameter values using the Plewes et al. method with an M_{tc} of 1.2 is markedly different from all three other methods. This can be seen by comparing the median value by this method (-0.108) to the medians of the other three methods (-0.048 to -0.017). This is especially important as the traditional Plewes method would have indicated that on average the sand tailings tested are not above the threshold of Ψ = -0.05, and therefore not likely susceptible to liquefaction. For this reason, using this method for screening without at least a lab M_{tc} may require additional judgement on the part of the engineer for what threshold should be set for identifying potentially liquefiable materials. A higher value of M_{tc} could also be assumed or estimated based on CPT correlations and other empirical relationships. The uncertainty when using Equation 8 to estimate λ_{10} has been previously shown through a wider range of data points including tailings (Reid 2015) and should also be considered before using the Plewes et al. method.

Second, while there is some variability between the results for Plewes et al. with lab values, Robertson, and Jefferies and Been, the general distributions are quite similar. Visually the Jefferies and Been method has a slightly higher median and perhaps a slightly wider

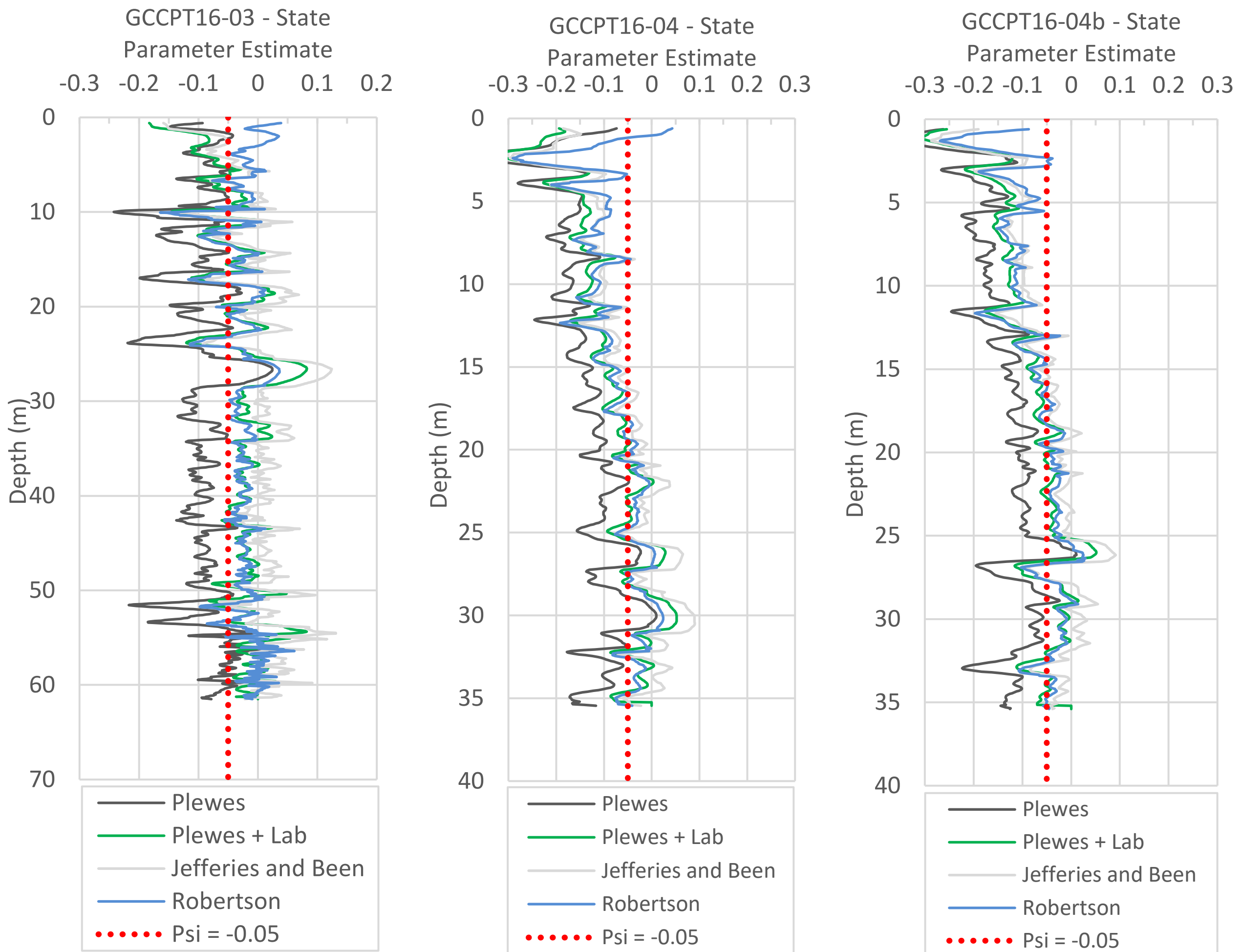

Figure 5. Distribution of state parameters with depth calculated by various methods for GCCPT16-03, GCCPT16-04, and GCCPT16-04b

distribution. The computed standard deviation for the Jefferies and Been method was 0.072 compared to 0.66 and 0.057 for the Plewes et al. with lab values, and Robertson methods, respectively.

Third, Figure 5 shows that the Plewes et al. method with lab values, Robertson, and Jefferies and Been methods trend relative to each other quite consistently with depth. The Plewes et al. with lab values and Robertson methods consistently agreed at all depths, with the Jefferies and Been method being slightly higher (more contractive) than both, especially at greater depths. This is consistent with the result of the distributions shown in Figure 4. The agreement between the Plewes et al. with lab values trend and the Robertson trend is encouraging as the Robertson evaluation can be completed without the testing required to obtain λ_e and M_{tc}. The variation when comparing these screening level methods to the Jefferies and Been method does show that there is value in performing the full calibration to obtain NorSand parameters for high risk projects.

Fourth, the slight differences between these methods emphasizes the importance of using a given method as a "package deal". For example, if someone wants to estimate state parameters using the Robertson (2010) method, and then take the concept of the 80th percentile being representative of the soil mass from Jefferies and Been (2016), they would have a much larger representative state parameter (-0.014) for use in design compared to the mean (-0.040) which is represented by Robertson. This could impact the residual strength estimate and even the assessment of liquefaction susceptibility. For this reason, it is essential that a consistent approach is used when assessing the liquefaction susceptibility and parameter estimation. As an aside on this topic, Robertson (2010b) recommends a threshold of $Q_{tn,cs} < 70$ to identify materials susceptible to liquefaction rather than basing this directly on calculated state parameter, but for the purpose of comparing to other methods, $Q_{tn,cs}$ was converted to state parameter using Equation 12.

Finally, the post-liquefaction undrained strengths (S_r/σ_{vo}') calculated based on these various methods are also very important for assessing liquefaction susceptibility. This is not discussed in detail, but it is worth comparing the residual strength estimates based on the Robertson method and the Jefferies and Been methods. The mean $Q_{tn,cs}$ from the Robertson method was used with the equation from Robertson (2010), based on a selected case history data set and yielded a residual strength ratio of 0.13. Taking the 80th percentile state parameter calculated by the Jefferies and Been method and the equation for residual strength given in Jefferies and Been (2016), which is based on a slightly different range of case history data, resulted in a residual strength ratio estimate of 0.06. It is worth noting that the residual strength ratio estimate given by Morgenstern et al. (2016) was 0.07. This value was based on additional site investigation data compared to what was analyzed above and used the Sadrekarimi (2014) correlations for strength estimation. Based on this comparison, careful consideration of methods should also be taken when estimating residual strength for design.

5 CONCLUSIONS

The estimation of state parameter based on the interpretation of CPT testing results is a critical element of some methods of static liquefaction susceptibility analysis. A brief history of the theory and development of these methods was presented to show the different ways that they approach the problem. The different data required for each, specifically lab testing data, was highlighted.

CPT data from the Fundao Tailings Dam report were digitized and then analyzed using the various interpretation methods to generate four different estimates of state parameter. The distribution of the results for each method were compared to show the differences in the entire data set. The results of each method were also compared with depth for three CPTs.

The authors found that the Plewes et al. method without any lab data consistently estimated lower (i.e. less conservative) state parameters for the sand tailings that were analyzed compared to the other methods. This led to the conclusion that careful consideration should be taken before implementing this method as a screening level analysis.

Another learning from this comparison is the differences observed in analysis that could be used at a screening level (ie no lab data needed). The difference in estimated state parameters between the Plewes et al. and Robertson methods showed that in most cases, assessment by more than one method at a screening level may be a prudent practice. Assessment by multiple methods is a marginal amount of additional work in data processing and could be a critical piece of data informing the hypothesis of whether a given soil is contractive or dilative.

The overall conclusion of this paper is that there are reasonably large differences between the state parameters calculated by the different methods trialed. This variability does make the difference between a conclusion of contractive vs dilative soils in some cases. The potential for misclassification based on the method selected could have significant stability implications, therefore the selection of method for estimating state parameter from CPT data should not be taken lightly.

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