Interpretation of the in situ density from seismic CPT in Fraser River sand

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ABSTRACT: The sandy deposits along the Fraser River delta in British Columbia, Canada tested for CANLEX provide a rare combination of reliable in situ measurements and laboratory element testing on undisturbed and reconstituted samples, allowing for direct evaluation of the capability of analytical methods in obtaining ground truth.
This work presents an analytical procedure to obtain the state parameter from CPT tip resistances in the CANLEX dataset. It includes calibration of a critical state constitutive model through triaxial compression tests, and analysis of the cone penetration using the spherical cavity expansion analogy. The effects of differing gradations and soil fabrics have been captured and reflected in the resultant state parameter interpretation. Accuracy is evaluated by comparison to in situ density measurements and comparison to other methods.

1 INTRODUCTION

With the recent advances in the analysis and design techniques used in geomechanics, accurate interpretation of 'ground truth' has become of even greater significance. In the case of cohesionless soils, 'ground truth' includes knowledge of in situ gradation, density, fabric and stress state, and the spatial variability of these parameters.

The behavior of cohesionless soils depends strongly on their density. While relative density, D_r, is an almost universally used density index for sand, it is easily shown that D_r can be misleading (e.g. Tavenas 1973). An alternative to D_r that captures the effects of both void ratio and mean stress on soil behavior is the state parameter, ψ (Been & Jefferies 1985). State parameter is defined as the difference between the current void ratio of the soil and its critical void ratio at the same mean effective stress. However, determining the in situ ψ (or D_r) in the laboratory is very difficult because of density changes during sampling. Penetration tests have thus become the norm for testing cohesionless soils, with the modern electronic CPT offering continuous data measurement with excellent repeatability and accuracy at relatively low cost.

The difficulty with any penetration test, however, is that the state value of interest is not measured. Instead it is calculated from the penetration resistance; a process usually referred to as interpretation. This interpretation involves solving an inverse boundary value problem to obtain mechanical properties from the measurements.

Ghafghazi & Shuttle (2008) analyzed a database of nine soils, including laboratory standard and natural sands, as well as relatively clean sand-size tailings, for which both chamber testing and triaxial compression data were available in the literature. This methodology offers a framework for interpreting the state parameter from CPT tip resistance.

The interpretation framework is applied to the Massey Tunnel site, an extensively investigated site in Fraser River Delta in Canada's British Columbia. The effect of soil fabric on the interpretation results has been considered by adjusting the calibration parameters with respect to tests on undisturbed samples. The accuracy of the method is evaluated by comparison to in situ density measurements and compared to other methods of interpreting the state parameter from CPT.

2 SITE INVESTIGATION PROGRAM

The Canadian geotechnical community completed a major collaborative research project between 1993 and 1997 entitled the Canadian Liquefaction Experiment (CANLEX). The project was divided into different phases. One of the sites investigated in Phase II was located south of Massey Tunnel, connecting Richmond and Delta in British Columbia. Complete summary reports with all data were published in a five volume series; the data for the Massey site reported in this paper are extracted from volume 4 (Wride & Robertson 1997). The site characterization program was targeted at depths of 8 to 13 *m* and included two standard penetration tests (SPT), six seismic cone penetration tests (SCPT), three self-boring pressuremeter tests (SBP), and two geophysical logs. Ground freezing and sampling was carried out, providing undisturbed cores which were trimmed into samples used in both triaxial and simple shear testing. Samples were also obtained using the Laval large diameter sampler.

The water table was measured at 2.3 *m* depth. γ_{sat} and γ_{dry} were 18.2 kN/m^3 and 13.4 kN/m^3 respectively. Based on SBP tests Wride & Robertson (1997) suggested the coefficient of lateral earth pressure at rest of k_0 = 0.5 for the target depth range; this evaluation has been adopted here. The frozen and Laval large diameter samples yield an average void ratio of 0.96 with standard deviation (SD) of 0.05.

3 MATERIAL AND TESTING

Fraser River Sand (FRS) is a uniform, angular to sub-angular with low to medium sphericity medium grained clean alluvial sand widely spread in the Fraser River delta. For Massey samples e_{min} and e_{max} are reported as 0.677 and 1.056 and $G_s = 2.68$.

Laboratory testing of the Massey site samples included testing reconstituted samples and undisturbed samples to evaluate the soil response to both undrained monotonic and cyclic loading. Since this method requires drained triaxial compression tests over a range of stresses and densities, which were not available in the CANLEX database, a second set of data on a batch of FRS entitled the "UBC sample" has been used. The fines content has been reduced to 0.8% to produce a clean sample. e_{min} and e_{max} are reported as 0.627 and 0.989 and G_s= 2.719 by Shozen (1991). Figure 1 illustrates the gradation curves of the two FRS samples used in this work.

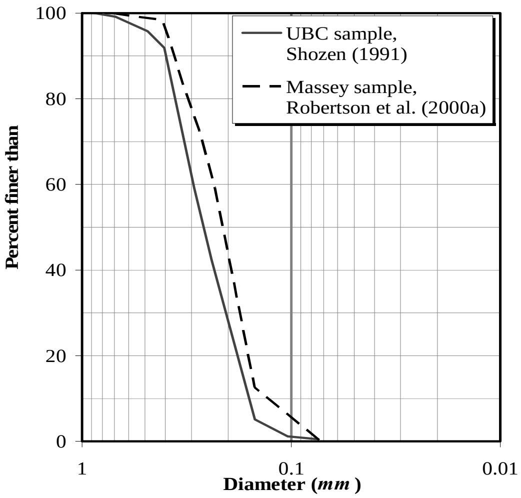


Figure 1. Gradation curves of Fraser River sand: UBC and Massey Samples

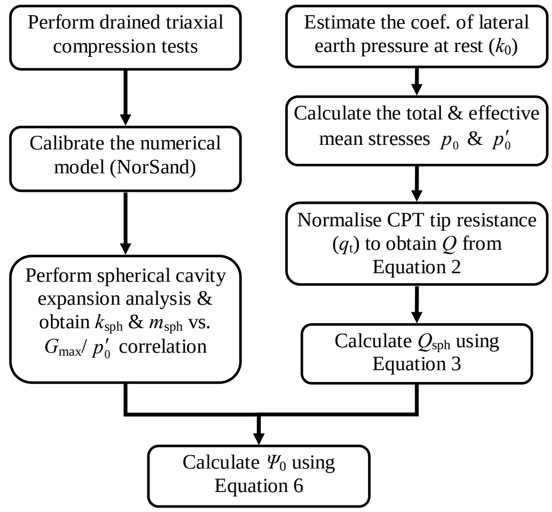


Figure 2. Flowchart for Ghafghazi & Shuttle (2008) method

4 METHODOLOGY

The CPT in sand provides just two signals; tip resistance (q_t) and sleeve friction; the penetration is drained so the pore pressure transducer simply measures hydrostatic pore pressure. The state measure used in this work is the state parameter, ψ. Because ψ is used as an internal state variable in the numerical model, the subscript '0' is used to denote the in situ (or initial) value of ψ_0 under geostatic conditions.

Initial work with determining ψ_0 from CPT data comprised triaxial testing of sands for which chamber test data was available to define their critical state locus (CSL), and then processing the chamber test data to develop dimensionless relations (Been et al. 1987) of the form:

$$Q = k \exp(-m\psi_0) \tag{1}$$

where Q is the normalized tip resistance defined as:

$$Q = \frac{q_t - p_0}{p'_0} \tag{2}$$

where p_0 is the initial mean total stress, and p'_0 is the initial mean effective stress. The two coefficients k and m in Equation 1 differ from one sand to another.

Since all analytical methods of interpreting ψ_0 from CPT must be verified against available calibration chamber data, the scatter in experimental results limits the accuracy of the interpretation method. Experimentalists have measured reproducibility of calibration chamber results by repeating tests on samples with the same density and stress conditions. These efforts have resulted in ± 25% error in measured Q in recent works (Hsu 1999). However, the majority of available data suggest that ±50% accuracy in measured Q is 'good' quality data. Of course, translating accuracy in Q to accuracy in ψ_0 includes some sort of interpretation, but a rough estimation based on average k and m values suggests that ± 0.05 is about the best accuracy that can be ex-

Table 1. Norsand calibration parameters for Fraser River Sand

	Critical State			Plasticity			Elasticity	
Parameter	Γ	λ_e	M_{tc}	H	χ_{tc}	N^*	$I_r = G_{\max}/p'_0$	ν
UBC (moist tamped)	1.22	0.060	1.45	80-310 ψ_0	3.2	0.45	$14.89\frac{(2-e)^2}{1+e}\times\left(\frac{p'_0}{p_r}\right)^{0.42}$ †	0.2
Massey (undisturbed)	1.17	0.035	1.49	110-310 ψ_0	3.2	0.45	Seismic CPT (650< I_r < 800)	0.2

† p_r is a reference pressure of 100 *kPa*

pected in prediction of any particular test in the available calibration chamber data.

Ghafghazi & Shuttle (2008) analyzed a total of 301 calibration chamber tests and achieved error levels of less than ±0.04 with 78% confidence level and less than ±0.07 with 92% confidence level. Their results were improved to 84% and 97% respectively for cases where elasticity was measured using bender elements. This is deemed to be an excellent accuracy achievable in an analytical method. This method is used here for evaluating the state parameter at the Massey site.

The method involves two parallel tasks: the normalization and processing of CPT tip resistance data, together with identification of soil behavior and calculation of material specific correlations. The correlations are then used to calculate the state parameter. Figure 2 presents a summary of the method and the required steps.

Normalizing and processing the CPT tip resistance data is done by estimating the stress state along the depth of interest. The total vertical and effective stresses can be calculated by estimating dry and saturated soil densities. The coefficient of earth pressure at rest (k_0) should be estimated or measured from in situ tests such as SBP or dilatometer tests. The mean stress can then be calculated and the normalized tip resistance Q can be calculated from Equation 2.

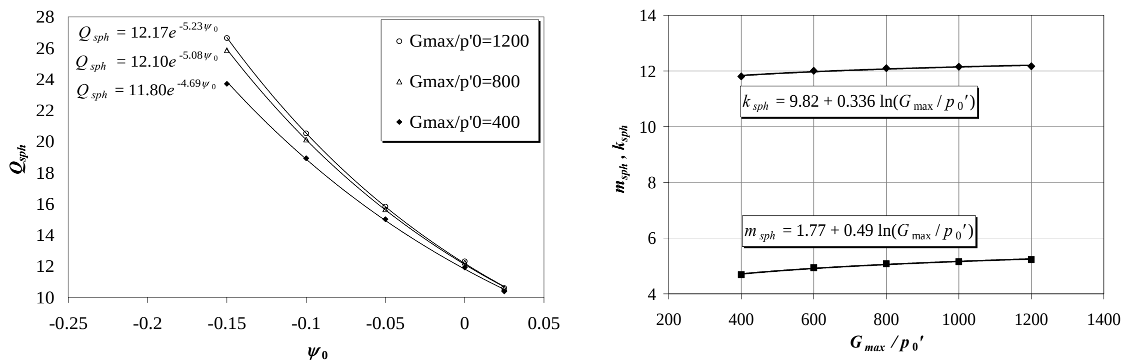
Central to the method is the application of a shape function which converts the normalized tip resistance Q to its spherical cavity expansion analysis equivalent Q_{sph}. Ghafghazi & Shuttle (2008) provided Equation 3 for the conversion:

$$Q_{sph} = \left(\frac{Q}{0.7}\right)^{0.59} \tag{3}$$

4.1 *Constitutive Modeling*

The behavior of the material is captured through calibration of the constitutive model to drained triaxial compression tests. The calibrated model is then used in a spherical cavity analysis to calculate Q_{sph} .

The constitutive model adopted in this model is NorSand (Jefferies 1993; Jefferies & Shuttle 2002; Jefferies & Shuttle 2005), an isotropically hardening - isotropically softening generalized critical state model that captures a wide range of particulate soil behavior. The complete model calibration was based on nine moist tamped "UBC sample" drained triaxial compression tests. The Massey sample data included 22 triaxial undrained and one drained triaxial compression test on undisturbed (frozen core) samples. The undrained tests were used to identify the critical state line (Γ, λ_e), and the critical state friction ratio (M_{tc}). This allowed the calibration parameters to be



a) b)

Figure 3. a. Q_{sph} vs. ψ_0 for range of $I_r = G_{\max}/p_0'$; b. m_{sph} and k_{sph} vs. normalized shear modulus I_r

modified to match altering material behavior caused by changing gradation. An adjustment to the hardening parameter H, which is deemed to be related to the fabric of the soil, is made based on the drained test. The calibration parameters for the two samples are presented in Table 1.

4.2 *Spherical Cavity Expansion Analysis*

The spherical cavity expansion analogy idealizes the CPT as a cavity in an infinite uniform medium under an isotropic stress state, with the internal pressure of the cavity initially equal to p_0'. The cavity is monotonically expanded by increasing its radius until a limiting (constant) pressure is obtained. This idealization greatly simplifies the analysis because the spherical symmetry allows only radial displacements, in turn permitting a one-dimensional description of the problem. The spherical cavity finite element code developed by Shuttle and Jefferies (1998) is used in this study. The code and finite element mesh remained the same, hence retaining the verified large displacement performance of it.

4.3 *Inverse form for Interpretation of CPT*

Shuttle & Jefferies (1998) showed that Equation 1 may be used to recover ψ_0 from CPT data provided that k and m are functions of soil characteristics and the stress level. Using a spherical cavity expansion analysis, we can summarize the effect of different soil characteristics in the form of NorSand parameters as:

$$k_{sph} = f_1(G_{\max}/p'_0, M_{tc}, N^*, H, \chi_{tc}, \lambda_e, \nu) \tag{4.a}$$
$$m_{sph} = f_2(G_{\max}/p'_0, M_{tc}, N^*, H, \chi_{tc}, \lambda_e, \nu) \tag{4.b}$$

All the NorSand parameters in Equation 4 are constants, or known functions of ψ_0. Hence at a particular ψ_0, all the variables in Equation 4 take a single value except for $G_{\max}/p'_0$ which is usually a function of both void ratio and stress level (the stress level effect). This makes Q_{sph} a function of the stress level at a particular ψ_0 (Figure 3a). k_{sph} and m_{sph} can then be determined as functions of $G_{\max}/p'_0$ (Figure 3b).

Having the normalized tip resistance Q, Q_{sph} can be determined from Equation 3,

Table 2. ψ_0 interpretation summary

Interpreta-tion method	Undisturbed sampling	Current work	Konrad (1997)	Been et al. (1987)	Plewes et al. (1992) †	Plewes et al. (1992)
average ψ_0	-0.055	-0.067	-0.114	-0.092	-0.089	-0.071
SD	0.050	0.028	0.032	0.021	0.038	0.038

† M_{tc} = 1.20 is used in the formula

and k_{sph} and m_{sph} from Figure 3b, provided that the shear modulus $G_{\max}$ is independently available. The in situ state parameter can then be calculated from Equation 5 which is directly deduced from writing Equation 1 for spherical cavity expansion:

$$\psi_0 = \frac{-\ln\left(Q_{sph} / k_{sph} \right)}{m_{sph}} \tag{5}$$

5 ANALYSIS AND RESULTS

Testing at the Massey site included a frozen sampling core surrounded by six CPTs at a radius of approximately 5 *m*. The Laval Large Diameter Sampler coring was also located on the perimeter of the layout. Although the target zone is identified as a fairly uniform layer, and the tests are relatively close, the CPT tip resistance measurements fall within a range between approximately 4 to 8 *MPa*. The range plotted in Figure 4a suggests lower values from 8 to 9 *m* depth and relative uniformity between 9 and 13 *m*. Parts of the logs are ignored between 12 and 13 *m* due to their discrepancy with the trend established by the rest of the tests. The normalized tip resistance Q is also plotted in Figure 4b as a range.

Considering k_0 = 0.5 and using the average $G_{\max}$ values obtained from seismic CPT measurements ($40MPa < G_{\max} < 70MPa$), a range of ψ_0 is calculated (Figure 4c). Ghafghazi & Shuttle (2008) margins of error of ±0.04 and ±0.07 are also plotted.

The results obtained from the methods proposed by Konrad (1997), Been et al. (1987) and Plewes et al. (1992) are plotted in Figure 4d. To be able to directly compare methods, the average and standard deviation (SD) of ψ_0 data obtained from each method are summarized in Table 2.

6 DISCUSSION

The samples trimmed from the cores obtained from ground freezing and Laval large diameter sampler are considered to represent the real in situ void ratio. An average void ratio of 0.96 with SD of 0.05 is obtained, translating into ψ_0 of -0.055 with the same SD. The wide scatter in the measured void ratios covers a range of $-0.155<\psi_0<0.01$. The scatter likely stems from the ground sampling techniques, rather than being a ground characteristic, as widely ranging void ratios were measured in samples taken from adjacent points of the core. And while the variation in measured void ratios represents loose to dense sand behavior, the geology of the site and in situ testing (including CPTs) imply a relatively uniform deposit. This paradoxical observation calls for more cautious treatment of the undisturbed sample results and emphasizes the need for better interpretation techniques for tests such as CPT.

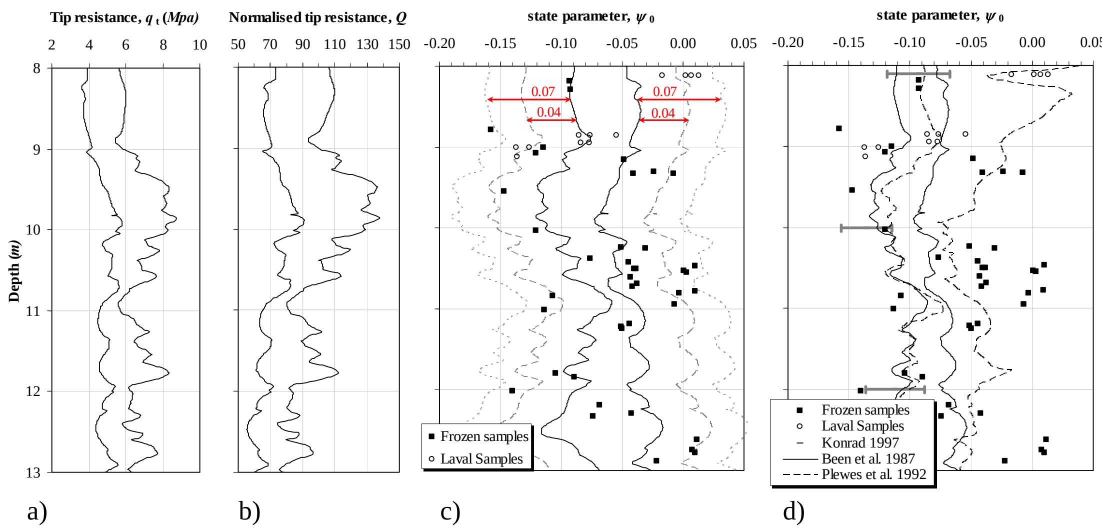


Figure 4. Upper and lower bound CPT response and state parameter interpretation for the target zone: a. Tip resistance b. Normalized tip resistance c. State parameter Interpretation (Ghafghazi and Shuttle, 2008) with ±0.04 and ±0.07 error margins d. Alternative methods of interpretation: Konrad (1997), Been et al. (1987), and Plewes et al. (1992)

As illustrated in Figure 4c, 98% of the undisturbed sampling measurement points fall within the ±0.07 error margins of the Ghafghazi & Shuttle (2008) method and 70.5% within the ±0.04 error margins. 20.5%, fall within the upper bound and lower bound lines, representing a zone of ideal accuracy for the information used in this work. The accuracy offered by the ±0.07 error margins covers a wide range of possible state parameters (typically $-0.3 < \psi_0 < 0.05$) when combined with the variation in the original CPT data represented by upper and lower bound envelopes. However, the range is very similar to that covered by the frozen and LDS samples, suggesting that this method is as capable as the most expensive and cumbersome of ground sampling techniques for determining the soil's in-situ density. The average ψ_0 of -0.067 is also very close to that measured by ground sampling. The difference is close to ±0.01; the ground sampling technique error margin given by Wride & Robertson (1997).

As shown in Table 2, Ghafghazi & Shuttle (2008) method provides the closest estimation for ψ_0 in this site. With the exception of Plewes et al. (1992), all interpretation methods presented in Table 2 require knowledge of the critical state line location in $e - \log p'$ space. The only additional requirement of the Ghafghazi & Shuttle (2008) method is for these tests to be performed under drained conditions; a requirement that does not pose any additional laboratory testing effort.

Plewes et al. (1992) correlated the slope of the critical state line in $e - \log p'$ space to the CPT friction ratio based on experimental results, hence eliminating the need to experimentally obtain the CSL. However, the method does require laboratory testing to measure the critical state friction angle (or the analogous M_{tc}). Plewes et al. (1992) suggested using $M_{tc} = 1.2$ ($\phi_{cv} = 30^\circ$) for all soils, advising that doing so would cause less than 10% error in the estimated ψ_0. Doing so puts the estimated ψ_0 in line with the other methods, resulting in a more negative (denser) state parameter than that implied by ground sampling techniques.

7 CONCLUSIONS

The success of different methods in obtaining the in situ state parameter appears to be directly related to the level of material behavior taken into consideration. Konrad (1997) does not account for any mechanical aspects of soil behavior and returns the most discrepancy in estimated state parameter. The Been et al. (1987) method accounts for material compressibility through the slope of the CSL. The Plewes et al. (1992) adds the effect of the critical state friction angle to their framework resulting in an even better estimation. The Ghafghazi & Shuttle (2008) analytical procedure also accounts for both compressibility and friction angle, and importantly adds elasticity, as well as stress level, dilatancy, and fabric.

An overall comparison of the model parameters for Fraser River sand to those of other sands presented in Ghafghazi & Shuttle (2008) suggests that the most important factor that makes the other methods systematically biased towards a more negative state parameter in Fraser River sand is its high critical state friction angle. This is further confirmed by the fact that a correct M_{tc} value in the Plewes et al. (1992) method results in a better interpretation.

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