Evaluation of cone penetration test data from a calcareous dune sand

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ABSTRACT: Cone penetration testing in calcareous soils presents an unusual case for the application of traditional correlations to soil properties based on siliceous sand behavior in that calcareous soils generally have higher friction angles than siliceous sands at the same state, although, much lower cone tip resistance. This paper evaluates CPT data from a calcareous dune sand through comparison of various in situ test results. Small strain stiffness parameters are higher than typical, while large strain stiffness parameters are typically lower. Normalization of tip resistance agrees with previous studies for siliceous sands, although high stress compressibility and the influence of soil state on $q_{c1N}$ (and $I_c$) lead to overprediction of fines content using conventional correlations.

1 INTRODUCTION

Evaluation of intra-correlations from a variety of in situ tests at a given site can provide useful information to calibrate theoretical models and empirical correlations. Downhole shear wave velocity ($V_s$) is useful for determination of small strain stiffness ($G_0 = \rho \cdot V_s^2$), self boring pressuremeter (SBP) tests provide measurement of shear moduli over intermediate strain levels, and the flat plate dilatometer test (DMT) may be used to assess a high strain stiffness value. The cone penetration test (CPT) is a versatile in situ test which can be performed more rapidly than the previously mentioned tests, and provides information of vertical variability in soil properties within a single sounding, or horizontal variability when comparing a number of soundings. This paper compares CPT, DMT, SBP, and $V_s$ data obtained in a calcareous aeolian sand located close to the top of a series of sand dunes near Ledge Point village, which lies about 100km north of Perth, Western Australia. The particular area of testing discussed here is 3.5 km from the Ledge Point beach site discussed by Hebeler et al. (2005), where the water table was much higher than at the present site. The sand at both sites is uniformly graded with $D_{50} \approx 0.24$mm and $e_{\text{max}} \approx 1.1$ to 1.3.
Figure 1. Cross section, location map and photo of Ledge Point dune site
Table 1. Summary of in situ test parameters for depth ranges and location of SBP tests

<table>
<thead>
<tr>
<th>ID</th>
<th>Mid Depth (m)</th>
<th>SBP</th>
<th>SCPT</th>
<th>CPT</th>
<th>DMT</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>p1.3% (kPa)</td>
<td>p4% (kPa)</td>
<td>s</td>
<td>G&lt;sub&gt;s&lt;/sub&gt;&lt;sup&gt;b&lt;/sup&gt; (MPa)</td>
</tr>
<tr>
<td>BH-A</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SCPT-9</td>
<td>1.3</td>
<td>100</td>
<td>168</td>
<td>0.46</td>
<td>20</td>
</tr>
<tr>
<td></td>
<td>2.3</td>
<td>292</td>
<td>486</td>
<td>0.45</td>
<td>52</td>
</tr>
<tr>
<td></td>
<td>3.3</td>
<td>501</td>
<td>833</td>
<td>0.45</td>
<td>84</td>
</tr>
<tr>
<td>BH-B</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SCPT-6</td>
<td>1.3</td>
<td>43</td>
<td>96</td>
<td>0.35&lt;sup&gt;a&lt;/sup&gt;</td>
<td>13</td>
</tr>
<tr>
<td>DMT-2</td>
<td>2.3</td>
<td>175</td>
<td>288</td>
<td>0.44</td>
<td>28</td>
</tr>
<tr>
<td></td>
<td>3.3</td>
<td>470</td>
<td>729</td>
<td>0.39</td>
<td>65</td>
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<tr>
<td></td>
<td>4.8</td>
<td>427</td>
<td>747</td>
<td>0.50</td>
<td>72</td>
</tr>
</tbody>
</table>

<sup>a</sup> This test appeared to be disturbed; the s parameter was fitted to data between ε<sub>c</sub> = 4 to 5%; calculation from p<sub>1.3</sub> and p<sub>4</sub> from Table 1 would result in s = 0.71

<sup>b</sup> The average of 3 unload-reload loops at different stress levels. Values did not vary significantly since the test were performed to the same stress ratio (e.g., Fahey 1991)

Figure 2. Typical profiles of CPT, DMT, SBP and SCPT parameters

2 TEST RESULTS

A test location plan, cross section, and photo of the site are shown in Figure 1. The test location plan indicates 5 CPTs, 3 additional SCPTs, 2 DMTs, 2 boreholes with self boring pressuremeter (SBP) tests, and two model pile tests (not discussed in this paper). Figure 2 plots results from SCPT-06, CPT-05, and DMT-02, all located in the southeast portion of the site near a trough between two dunes. Additionally, shear moduli inferred from V<sub>s</sub> for SCPT-06 are compared to average unload-reload modulus from the SBP tests in Borehole B. Characteristic SBP cavity pressure values for adjusted cavity strains of 4% are included with the DMT data. These data and data adjacent to Borehole A are also summarized at pressuremeter test depths in Table 1. It is noted that characteristic pressures for the SBP tests have had the creep component removed to enable appropriate comparison with rapid expansion induced by the cone or DMT.
Figures 1 and 2 indicate a relatively consistent soil type, with a friction ratio (F) of around 0.5% constant with depth. No significant changes in soil type were observed from borehole cuttings and test pits near CPT-08 indicate clean sands. Tip resistance in the cross section on Figure 1, as well as profiles in Figure 2, indicate two main sand layers that repeat throughout the vertical profile; one layer with a high tip resistance (discussed as ‘dense’ and indicated as darker color layers) and one layer with a lower tip resistance (discussed as ‘loose’ and indicated as lighter color layers). The loose and dense layers are interbedded due to the shifting nature of the sand dunes. Thicker deposits of loose sand layers tend to accumulate at the trough between two dunes (i.e., near CPT-8), thinning up the side of the dune. The predominant wind direction is typically W-E/E-W, or perpendicular to the direction of the cross section.

3 CORRELATIONS BETWEEN STRENGTH AND STIFFNESS

There is not a unique correlation between strength and stiffness for soils, but assessment of the correlation between measurements of stiffness and strength at a given location provides valuable information on the possibility of ageing, cementation, and suctions induced by partial saturation. Measurements of stiffness were obtained from seismic cone (G₀), self boring pressuremeter (Gₓr), and the dilatometer (Eₓ). When evaluating correlations between strength and stiffness based on in situ tests in sands, the ratio of G/qcnet (or E/qcnet) are often compared with an indication of relative density, such as the normalized cone tip resistance q₁N \[\frac{q}{q_{c\text{net}}} = \left(\frac{q_{\text{net}}}{p_{\text{ref}}}\right) / \left(\frac{\sigma'_{v_0}}{p_{\text{ref}}}\right)^{0.5}\] with \(p_{\text{ref}}=100\text{ kPa}\). This relationship can be expressed as:

\[
\frac{G_0}{q_{\text{cnet}}} = K_G \cdot \left(\frac{q_{\text{cnet}}}{p_{\text{ref}}}/\left(\frac{\sigma'_{v_0}}{p_{\text{ref}}}\right)^{0.5}\right)^{-n} = K_G \cdot q_{1N}^{-n}
\]

where \(p_{\text{ref}}\) is a reference stress equal to 100 kPa. Typical values of the stress exponent, \(n\), vary from 0.67 to 0.82, and a value of 0.75 is adopted in this paper (after Rix & Stokoe 1991). Theoretical studies of siliceous and calcareous sands as well as a data base of natural uncemented Holocene sand sites suggest that \(K_G\) on average is about 215. Studies have shown that \(K_G\) varies widely depending upon stress state, ageing, and cementation (e.g., Fahey et al. 2003, Schneider 2009), and for a stress exponent of 0.75 tends to range from 110 to 330 in uncemented Holocene siliceous sands, and from 330 to 1100 in aged sands, cemented sands, and residual silty sands. Figure 3 compares the ratio of \(G_0\) to \(q_{\text{cnet}}\) to these ranges for the Ledge Point sand. It is apparent that data follow the conventional correlation, with \(n\) of 0.75, and that the average \(K_G\) value of about 500 is higher than typical of uncemented unaged sands. By comparison, laboratory values of \(G_0\) from bender elements tests on Ledge Point sand when combined with conventional relative density correlations indicate a \(K_G\) value of approximately 400. This would imply that both the ‘loose’ and ‘dense’ natural sands at Ledge Point have a stiffness value that is 25% higher than reconstituted samples, indicating some degree of cementation or bonding. High \(K_G\) values at Ledge Point may also arise due to suctions present in the sand, which was at a typical saturation level of less than 30%.
The ratio of the dilatometer modulus ($E_D$) to CPT $q_{cnet}$ shown in Figure 3 has a median value of 2.7, and no density ($q_{c1N}$) dependence. The lack of density dependence is typically observed for $E_D/q_{cnet}$ in siliceous sands, although the characteristics value of $E_D/q_{cnet}$ is usually closer to 4.

Figure 3. Comparison of stiffness and strength

4 CONE TIP RESISTANCE AND SOIL CLASSIFICATION

Cone penetration testing in saturated Ledge Point calcareous sands (e.g., Hebeler et al. 2005) produced no excess penetration pore pressures (i.e., $u_2-u_0$), indicating conditions of drained penetration. The CPTs and SCPTs in the Ledge Point dune sand considered here were located well above the water table, and penetration conditions are also considered drained for this case. In cases of drained penetration, comparisons of relative variations in normalized cone tip resistance to variations in friction ratio (F), or soil behavior type index [$I_{C}=f(q_{c1N},F)$; e.g., Robertson & Wride 1998], can provide more evidence of potential changes in soil behavior than comparison of normalized cone tip resistance and pore pressure parameters. Soil classification in this paper is based on normalized cone tip resistance and friction ratio, as discussed by Robertson (1990) and Robertson & Wride (1998).

While cone tip resistance is more strongly influenced by horizontal effective stress, normalized cone tip resistance is usually expressed in terms of vertical effective stress for ease of calculation:

$$\left(\frac{q_{cnet}/p_{ref}}{\sigma'_{vo}/p_{ref}}\right)^n$$

where $p_{ref}$ is a reference stress equal to 100 kPa and $n$ is a stress exponent (which differs from that discussed for relationships between strength and stiffness). Traditionally, $n$ values of one have been used and the normalized cone tip resistance is identified as $Q (=q_{cnet}/\sigma'_{vo})$. When analyzing relative density and friction angle of sands,
stress exponents of 1, 0.71, 0.55, 0.6, and 0.5 have been used by various authors. The latter has been adopted for evaluation of liquefaction resistance and soil classification in sands, and is often known as (the previously defined) \( q_{c1N} \). The stress exponent for sands may actually vary with relative density due to stress dependent dilation, which is explored for these sands through results in Figure 4.

As previously illustrated in Figures 1 and 2, the shifting nature of the formation of sand dunes has led to locations of comparatively ‘dense’ and ‘loose’ layers. When data from the 9 CPTs and SCPTs are compiled (for a depth range of 0 to 12m, 4300 points), a bi-modal distribution is observed in Figure 4a. The ‘loose’ layers tend to have a characteristic value of \( q_{cnet} \) near 8 MPa, while the denser layers tend to have \( q_{cnet} \) near 24 MPa. Histograms for \( Q \), \( q_{c1N} \), and optimized stress exponents of 0.45 for the ‘dense’ sand layers and 0.55 for the ‘loose’ sands are illustrated in Figures 4. The optimized stress exponent was selected based on the minimum COV of normalized cone resistance that occurred as \( n \) was increased from 0 (\( q_{cnet} \)) to 1 (\( Q \)). For this case, over a depth range of 0 to 12m, it is true that the optimum stress exponent appeared to be slightly higher for loose calcareous sands (0.55) than for dense calcareous sands (0.45). Both stress exponents were approximately equal to 0.5, and median (or average) values were within about 2 percent. These uncertainties are small compared to uncertainties due to model error and vertical and horizontal variability in the soil deposit itself. A stress exponent of 0.5 (\( q_{c1N} \)) would therefore seem most applicable for analysis of normalized behavior for these calcareous sands.

While use of a stress exponent of 0.5 produces the most uniform normalized behavior for these calcareous sands, it may not be most appropriate for assessment of soil classification by CPT. For analysis of clays, a stress exponent of unity (\( Q \)) is most commonly used. If different stress exponents are used for sands and clays, this requires an iterative procedure which is slightly more complicated and may not improve the analysis a great deal. Figure 5 compares assessment of soil behavior type using \( Q-F \) and \( q_{c1N}-F \) charts. Charts are based on Robertson (1990) and Robertson & Wride (1998), although the areas of ‘Sensitive’ and ‘Overconsolidated / Cemented’ have been replaced by a slightly larger version of the originally recommended ‘Normall Consolidated’ Zone.

When using either \( Q \) or \( q_{c1N} \), the same conclusions are made in terms of classification for these soils. Soils classify predominantly in Zones 6 (sands), with a few points in Zones 7 (gravelly sands to sand) and 5 (silty sand to sandy silt). More points plot in Zone 7 when using \( Q \) as compared to \( q_{c1N} \), which may relate more to tip resistance being controlled by soil state (relative density and effective stress) as compared to relatively density alone (e.g., \( q_{c1N} \)). The parameter \( I_c \) increases as the soil type moves from Zone 7 to 6 to 5, etc. Figure 4 illustrates histograms of friction ratio (\( F \)) and \( I_c \). The soil has a very uniform friction ratio at about 0.5%, but reductions in \( q_{c1N} \) due to soil state leads to increases in \( I_c \). Values of \( I_c < \) than 1.31 are usually considered sands and gravels (Zone 7) and as \( I_c \) increases it is inferred that fines content increases to a value of 15 to 30% nonplastic fines at an \( I_c \) of 2 (e.g., Robertson & Wride 1998). Visual observations in test pits near CPT-08 (i.e., a ‘loose’ zone) did not show high fines content materials. The compressible nature of calcareous sands may lead to overestimation of fines content using correlation developed for siliceous sands. Additionally,
investigations into the influence of soil state on $I_c$ for interpretation of fines content in siliceous and calcareous sands are clearly warranted.

Figure 4. Frequency of distributions for CPT parameters (8 CPTs; 4300 data points; depth = 0 to 12m)

Figure 5. Location of calcareous sands in soil behavior type charts slightly modified after Robertson (1990) and Robertson & Wride (1998) (8 CPTs; 4300 data points; depth = 0 to 12m)
5 SUMMARY & CONCLUSIONS

Comparison of various in situ test results at a given site provide additional insights into soil behavior. The cone penetration test provides an abundance of near continuous data with depth in a rapid and repeatable manner, but understanding of whether high and low cone tip resistance values are influenced by changes in density, overconsolidation, cementation, or suctions requires additional measurements such as those from downhole $V_s$ tests, SBP test, or DMT, as well as laboratory tests. This paper presented results of these various in situ tests in a calcareous sand dune with a bimodal cone tip resistance distribution, i.e., characteristically ‘loose’ and ‘dense’ interbedded sand layers.

For this site the following conclusions have been drawn:

- Both the ‘loose’ and ‘dense’ sand layers had $G_0/q_{cnet}$ ratios that were in line with typical correlations having uniform values of ‘$K_G$’. These values are higher than uncedmented siliceous and calcareous sands.
- The values of SBP $G_{up}/G_0$ ratios were typically around 0.3 to 0.4, and dropped as low as 0.15 in near surface layers. These values are lower than values of 0.4 to 0.6 typically observed in siliceous sands.
- The $E_D/q_{cnet}$ ratio followed conventional trends but was also lower than average, possibly due to high stress compressibility or effects of cementation.
- Tip resistance normalization using stress exponents of 0.45 for ‘dense’ layers and 0.55 for ‘loose’ layers produce the most uniform distributions of normalized tip resistance. For practical purposes, a stress exponent of 0.5 leads to relatively consistent distributions of normalized tip resistance.
- Test pits did not indicate the presence of fines in soils identified as ‘loose.’ It is inferred that decrease in $q_{c1N}$ due to high stress compressibility and reduction in soil state result in increased values of $I_c$ and misinterpretation of the fines content. The influence of state on $I_c$ and inference of fines content requires additional study.

REFERENCES


