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Suggested QC criteria for deep compaction using the CPT

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ABSTRACT: The most common forms of deep compaction in sandy soils involve vibratory techniques, such as, vibro-compaction, vibro-replacement, dynamic compaction and rapid impact compaction. Historically, it was common to define quality control (QC) criterion for deep compaction based on a change in relative density (D_r). However, D_r cannot be directly measured in-situ at depth and does not always have well defined links with soil behavior, especially in sandy soils with high fines content. In more recent years the CPT has become popular for QC for deep compaction since it is; fast and cost effective, provides a continuous profile, gives re-liable and repeatable measurements and provides more than one measurement. Although it is possible to link CPT measurements with D_r the correlations are not unique, apply only to clean sands and can vary considera-bly with grain mineralogy and characteristics. Current methods of using CPT measurements for QC for deep compaction often apply only to clean silica sands and are not effective in soils with higher fines. This has fre-quently resulted in unnecessary disputes, delays and cost over-runs concerning the effectiveness of the deep compaction. A suggested approach for QC for deep compaction is described based on the normalized *equivalent clean sand* cone resistance ($Q_{tn,cs}$). A brief background on the development of $Q_{tn,cs}$ based on liquefaction case his-tories and how this parameter has been linked to in-situ State Parameter is provided. Hence, there is strong theoretical and experimental evidence that $Q_{tn,cs}$ is a measure of in-situ state and hence, linked to soil behavior and response over a wide range of soils from clean sand to low plastic silt.

1 INTRODUCTION

Ground improvement is often carried out for the following main reasons:

- Increase bearing capacity for shallow foundations (i.e. improve soil strength)
- Decrease settlements (i.e. improve soil stiffness)
- Increase resistance to earthquake loading (i.e. reduce soil liquefaction)

The most common forms of deep compaction in sandy soils involve vibratory techniques, such as, vibro-compaction, vibro-replacement, vibro-displacement, dynamic compaction and rapid impact compaction. These methods compact the soil through vibration that disrupts the sand structure to form denser packing. Vibratory methods also change the in-situ lateral stresses (increase K_o) but can also destroy any existing microstructure (e.g. aging, bonding, etc.).

Historically, it was common to define quality control (QC) criterion for deep compaction based on a change in relative density (D_r). However, D_r cannot be directly measured in-situ at depth and does not always have well defined links with soil behavior, especially sandy soils with high fines content.

In more recent years the CPT has become popular for QC for deep compaction since it is: fast and cost effective, provides a continuous profile, gives reliable and repeatable measurements and provides more than one measurement (e.g. cone resistance, q_c, sleeve resistance, f_s, penetration pore pressure, u_2 and sometimes shear wave velocity, V_s). Although it is possible to link CPT measurements with D_r the correlations are not unique and can vary considerably with grain mineralogy and characteristics (e.g. fines content).

Research linking either SPT (N value) or CPT penetration resistance (q_c) with D_r in clean silica sands has also shown that the penetration resistance varies with overburden stress (i.e. depth). In the past it was common to have QC criteria based on a single value of penetration resistance (either SPT N value, or CPT tip resistance q_c). Since penetration resistance varies with depth, due to increasing overburden stress, the approach of defining a single constant penetration resistance often creates problems when ground improvement cannot achieve the QC criteria at shallow depth due to the small overburden stress and hence, low penetration resistance. In more recent years it has become more common to provide QC criterion that has penetration resistance increasing with depth in an effort to capture the influence of overburden stress.

2 NORMALIZED PENETRATION RESISTANCE

In clean sands penetration resistance usually varies in a non-linear manner with depth that has resulted in the use of normalized penetration resistance, such as:

$$\text{SPT} \quad (N_1) = N\, C_N \tag{1}$$

$$\text{CPT} \quad q_{c1} = q_c\, C_N \tag{2}$$

Where:
(N_1) = normalized SPT blow count, corrected to an effective overburden stress of 1 atmosphere (σ'_{vo} = 1 atm. = 1bar ~ 1tsf ~ 0.1MPa = p_a). It has also become common to correct SPT N values to a 60% energy, to become $(N_1)_{60}$.

q_{c1} = normalized CPT cone resistance, corrected to an effective overburden stress of 1 atmosphere (σ'_{vo} = 1 atm. = 1bar ~ 1tsf ~ 0.1MPa = P_a). Often the normalized CPT cone resistance is made dimensionless to become:

$$q_{c1N} = q_{c1}/P_a \tag{3}$$

C_N = overburden correction factor.

Typically the correction factor (C_N) in clean sands is simplified to the following:

$$C_N = (P_a/\sigma'_{vo})^{0.5} \tag{4}$$

Hence, when $\sigma'_{vo} = P_a$ = 1 atm., C_N = 1.0.

Robertson (2009) suggested a more generalized and dimensionless form of normalized cone resistance, Q_{tn}, as follows:

$$Q_{tn} = \ [(q_t - \sigma_v)/P_a]\ (P_a/\sigma'_{vo})^n \tag{5}$$

Where:
q_t = cone resistance corrected for water pressure (Campanella and Robertson, 1982)
$(q_t - \sigma_v)/P_a$ = dimensionless net cone resistance
$(P_a/\sigma'_{vo})^n$ = overburden correction factor, C_N
n = stress exponent that varies with soil type.
σ_{vo} and σ'_{vo} are the total effective overburden stresses respectively;
P_a is a reference atm. pressure in the same units as σ'_{vo}, q_c and σ_{vo}.

Robertson (2009) suggested that the stress exponent (n) could be evaluated from the CPT soil behavior type (SBT) index, I_c, as follows:

$$n = 0.381\,(I_c) + 0.05\,(\sigma'_{vo}/P_a) - 0.15 \tag{6}$$

Where:
$I_c = [(3.47 - \log Q_{tn})^2 + (\log F_r + 1.22)^2]^{0.5}$
F = $f_s/[(q_c - \sigma_{vo})]$ x 100% is the normalized friction ratio (in percent)
f_s is the CPT sleeve friction stress;

In most sands, n = 0.5 and q_c is large relative to σ'_{vo}, then $(q_t - \sigma_{vo}) \sim q_c$, and $Q_{tn} \sim q_{c1N}$.

3 SOIL BEHAVIOR TYPE INDEX AND COMPACTABILITY

Massarsch (1991) showed that vibratory compaction methods tend to be more effective in clean sands and Degan et al (2005) showed that vibratory compaction methods become less effective with increasing soil behavior type (SBT) index, I_c as shown in Figure 1. Hence, I_c can be used as a guide on the potential effectiveness of many vibratory compaction methods. Typically, when I_c > 2.6 vibratory compaction methods become less effective, in terms of improved density as measured by penetration resistance. However, soils with I_c > 2.6 can have improved behavior characteristics due to changes in stress history and K_o.

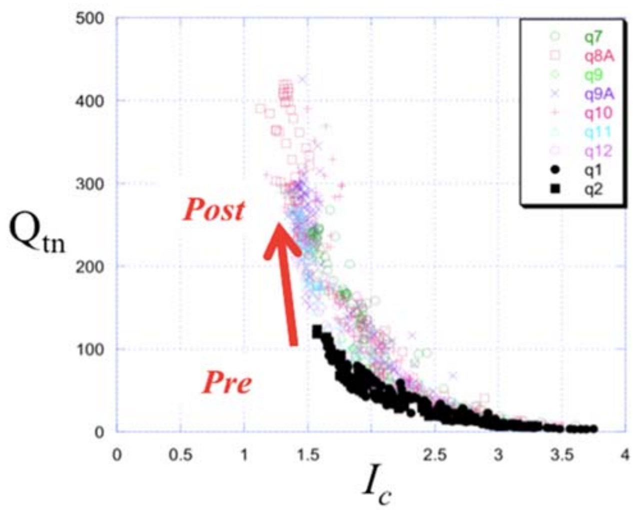

Figure 1. Change in normalized cone resistance, Q_{tn}, as a function of SBT I_c due to vibro-compaction (After Degan et al, 2005)

4 EQUIVALENT CLEAN SAND PENETRATION RESISTANCE

In clean sands a Q_C criteria based on either q_{c1N} or Q_{tn} is preferred over a simple linear variation of q_c, since it links better with relative density and soil response. Normalized values also have the advantage that they are based on vertical overburden stress (σ'_{vo}) that accounts for soil unit weight and ground water conditions at the time of the in-situ test rather than depth. Normalized penetration resistance (e.g. Q_{tn}) is preferred over a relationship of measured penetration resistance versus depth (e.g. q_c vs. depth) and can be effective in deposits of clean sand regardless of depth and groundwater level. However, not all soil profiles are composed of clean sands. Frequently, either natural soils or hydraulically placed fills are composed of varied layers of clean sand, silty sand, silt and sometimes clay. Experience and research has shown that penetration resistance is strongly influenced by the compressibility of soil. Hence, sand with high fines content have a lower penetration resistance than a similar clean sand even though both soils can have the same behavior/response under loading (e.g. similar strength and stiffness). The current practice of defining a QC criterion in terms of either a simple linear variation of q_c or normalized tip resistance (either q_{c1N} or Q_{tn}) will not be effective in the layers with higher fines content.

Experience based on many case histories, using penetration resistance to evaluate soil liquefaction has identified the benefits of correcting measured penetration resistance to an '*equivalent clean sand'* penetration resistance based on parameters such as fines content or CPT SBT index, I_c. Several methods are available to determine the '*equivalent clean sand'* normalized cone penetration resistance (e.g. Robertson and Wride, 1998; Moss et al, 2006; Idriss and Boulanger, 2008; Boulanger and Idriss, 2014). The simple method suggested by Robertson and Wride (1998) to calculate the '*equivalent clean sand'* normalized CPT cone resistance ($Q_{tn,cs}$) is based on the following:

$$(Q_{tn})_{cs} = K_c\, Q_{tn} \tag{7}$$

Where K_c is a correction factor that is a function of behavior characteristics (e.g. combined influence of fines content and plasticity) of the soil. Robertson and Wride (1998) recommended the following relationship between I_c and the correction factor K_c:

$$K_c = 1.0 \qquad \text{if } I_c \leq 1.64 \tag{8}$$

$$K_c = 5.581 I_c^3 - 0.403 I_c^4 - 21.63 I_c^2 + 33.75 I_c - 17.88 \qquad \text{If } I_c > 1.64 \tag{9}$$

Robertson (2010 and 2012) showed that $Q_{tn,cs}$ can be linked to the in-situ state of a soil using the State Parameter, ψ (Been and Jefferies, 1992). Hence, there is strong theoretical and experimental evidence that $Q_{tn,cs}$ is a measure of in-situ state and hence, linked to soil behavior and response over a wide range of soils from clean sand to low plastic silt. Since $Q_{tn,cs}$ is used to evaluate the resistance of soils to cyclic loading (i.e. liquefaction resistance), it has direct application when deep compaction is used to improve ground against earthquake loading. Given the link between $Q_{tn,cs}$ and State Parameter, it also has direct application when deep compaction is used to improve both strength and stiffness of soil for either increased bearing capacity and/or reduced settlement.

Figure 2 presents an example of a CPT profile in a typical ground profile that has varied soil conditions ranging from clean sand to silty clay and Figure 3 shows the same location before and after deep compaction using vibro-compaction. Figure 3 shows that a criterion based on normalized cone resistance (Q_{tn}) is unable to capture the behavior in the fine-grained soils between depths of about 6m to 10m (20 to 31 feet), whereas the clean sand equivalent normalized cone resistance ($Q_{tn,cs}$) is better able to capture the behavior for all the soils over the full depth profile. The example shown in Figure 3 was for a project where deep compaction was used to reduce the risk of soil liquefaction. The data shown in Figure 3 after compaction produced acceptable resistance to liquefaction under the project design earthquake.

5 SUMMARY AND RECOMENDATIONS

Unless soils are clean sand, it is recommended that the QC criterion for deep compaction should be based on CPT data using '*equivalent clean sand normalized cone resistance', $Q_{tn,cs}$*. This requires a pre-agreed method to calculate $Q_{tn,cs}$, since it depends on several factors, such as effective overburden stress and fines content. A method to calculate $Q_{tn,cs}$ was suggested by Robertson and Wride (1998) that has been shown to be simple and effective. A QC criterion based on $Q_{tn,cs}$ cannot be presented as a simple plot of measured cone resistance (q_c) versus depth, since it depends on the soil characteristics (e.g. fines content) at the location of the test. Fortunately, commercial software is available (or custom software/spreadsheets) that can calculate $Q_{tn,cs}$ quickly so that it can be used efficiently to provide effective QC for deep compaction methods.

A more detailed description of the above approach is provided in a recorded webinar that can be freely downloaded at: *www.greggdrilling.com/webinars/DwIgW/cpt-for-quality-control-of-ground-improvement-deep-compaction.*

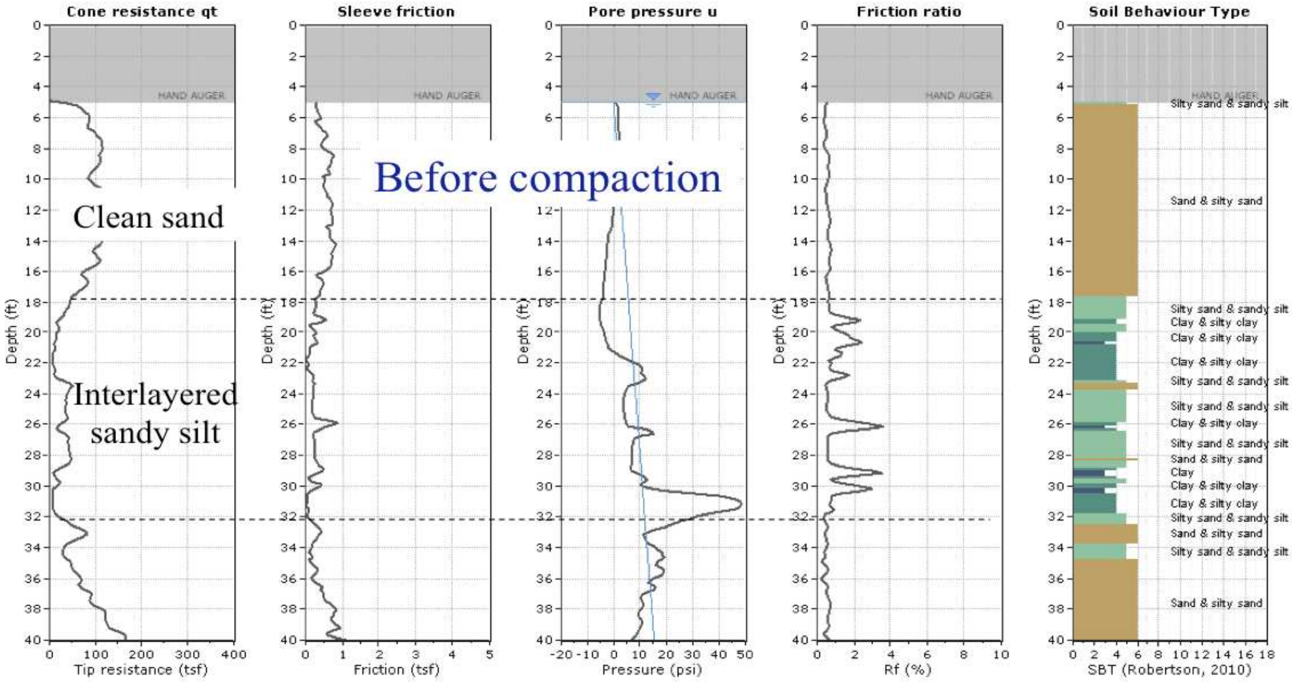

Figure 2. Example CPT profile (before compaction) in typical varied ground conditions
(note: 1 foot ~ 0.3m, 1 tsf ~ 0.1 MPa, 1psi ~ 6.7 kPa)

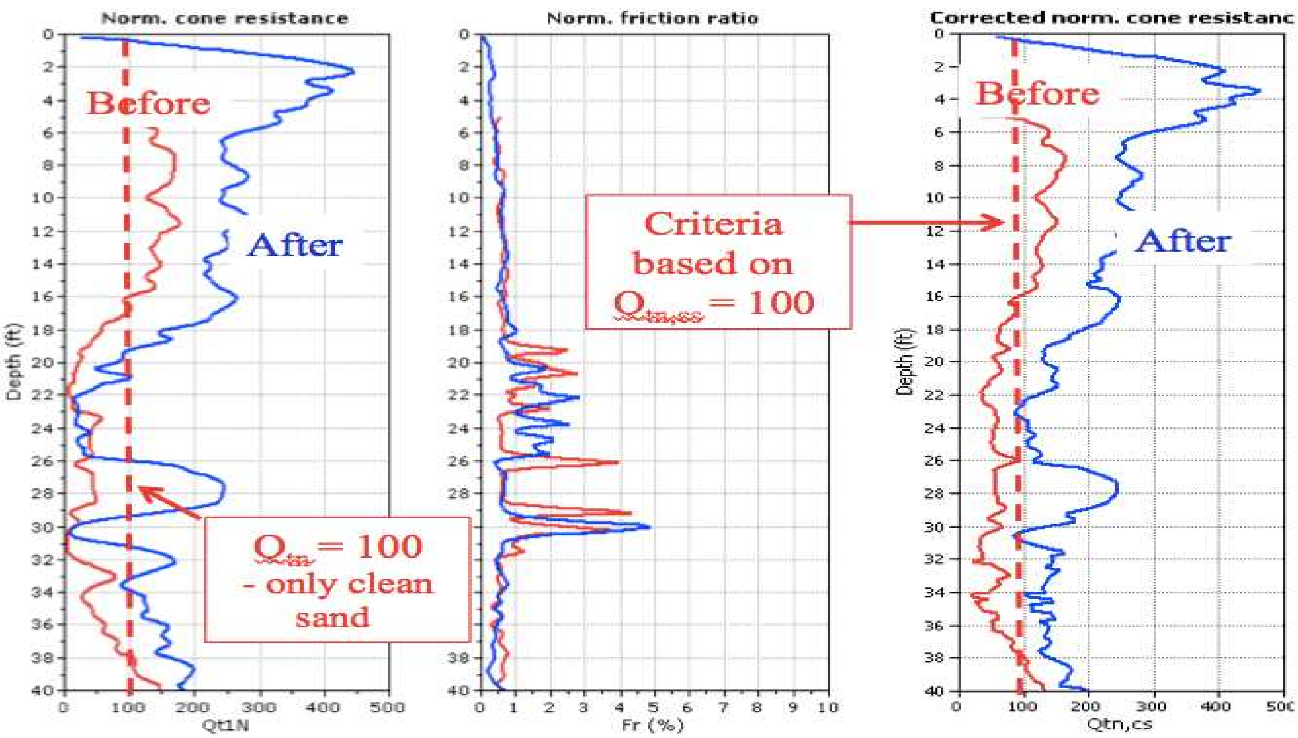

Figure 3. CPT profile at same location as data shown in Figure 2 showing before and after deep (vibro) compaction profiles in terms of normalized cone resistance (Q_{tn}) and equivalent clean sand (corrected) normalized cone resistance ($Q_{tn,cs}$) and the QC criterion of $Q_{tn,cs} > 100$.

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