LIQUEFACTION ANALYSIS USING THE CONE PENETROMETER TEST

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SUMMARY

This paper presents a new liquefaction analysis method using in-situ Cone Penetrometer Test (CPT) data. The first step is to develop CPT and cyclic laboratory data normalization techniques for rational data comparison at different depths and soil types. Then a technique is developed for predicting the Standard Penetration Test (SPT) N values using CPT data. The resultant is a normalized liquefaction resistance strength ratio (i.e., cyclic stress ratio) in terms of contours on a newly developed CPT Soil Characterization chart based on cyclic laboratory data trends and SPT liquefaction techniques using CPT-SPT trends.

INTRODUCTION

The liquefaction resistance strength ratio (i.e., cyclic stress ratio) is generally determined using the Standard Penetration Test (SPT) because it is a simple fast index test. If the project is critical or the SPT analysis indicates a marginal factor of safety, then cyclic laboratory tests should be performed. These two methods represent the extremes; a simple in-situ index test and an expensive involved laboratory technique. The Cone Penetrometer Test (CPT) is the likely replacement for the SPT because it can provide two repeatable unique measurements continuously with depth (Refs.1 and 2). The two CPT measurements are cone resistance (i.e., end bearing, q_c), and sleeve friction resistance (i.e., incremental side friction, f_s) generally presented in units of tons per square foot (tsf) together with the friction ratio which is the sleeve friction resistance divided by the cone resistance and multiplied by 100.

Developing the CPT liquefaction technique required the following: (1) an understanding of soil sensitivity, (2) development of a CPT normalization technique, (3) CPT-to-SPT correlation, (4) laboratory liquefaction data normalization, and (5) CPT to SPT to cyclic laboratory data correlation.

SOIL SENSITIVITY

The CPT f_s is a sensitivity-affected strength measurement for clays and soil mixtures because the soil is partially remolded during the passage around the CPT cone tip (Ref.1). Sensitivity can be defined as the ratio of in-situ undrained strength to the high strain (or remolded) strength. Clay with normal sensitive will have an f_s close to the undrained strength. A soil mixture can be very unstable due to a loose or honeycombed sand structure and underconsolidated clay matrix (Refs.1 and 3). This type of structure is created by a soil slurry consolidating to a stress at which the sand and silt grains are in full contact. With further stress increase, the clay matrix strength does not change (i.e. increasing underconsolidation) and the total

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mass becomes increasingly sensitive. Sensitive soil mixtures created from a slurry may have a lower liquefaction resistance compared to a comparable loose sand skeleton because it was created at a slower velocity. On the other hand, the strain potential of the sensitive soil mixture is likely to be lower than the comparable loose sand because of the clay matrix strength. CPT interpretation of soils having a sensitive structure is likely to be complicated by age effects and overconsolidation (or desiccation). Sands with relative densities less then about 40 percent have a lower than normal f_s and large dynamic strain potential which is important to understand for CPT liquefaction assessment. The assessment of soil sensitivity is important because conceptually a soil could have a constant liquefaction potential if the in-situ strength and soil sensitivity increases simultaneously (i.e., increasing q_c and decreasing f_s). Therefore, a liquefaction resistance index might be thought of as the combination of static strength and the difference between the static strength and the high strain (or remolded) strength.

CPT NORMALIZATION

The conventional method for CPT normalization uses the CPT friction ratio and the C_p factor (Ref.2). The friction ratio has no direct engineering application and is difficult to interpret for soil classification in soft, sensitive and overconsolidated soils (Ref.1). The C_p factor is an empirical relationship for converting the q_c to an equivalent value at a effective vertical stress of 1 tsf in sand and is the same type of relationship as the C_n factor (Ref.4) which was developed for normalizing the SPT. The C_p (or C_n) relationship can be defined with an exponential n value (Equation 1) and used to normalize the CPT q_c to q_{cn} (Equation 2). The general bearing capacity formula (Equation 3) can be related to q_{cn} for clay and sand as shown in Equation 4.

$$C_n = C_p = \frac{1}{[\bar{\sigma}_v]^n} \quad (\text{ for } \bar{\sigma}_v < 5 \text{ tsf }) \quad \ldots\ldots\ldots\ldots\ldots\ldots \; 1$$

$$q_{cn} = q_c\, C_n = \frac{q_c}{[\bar{\sigma}_v]^n} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \; 2$$

$$q = cN_c + \bar{\sigma}_v N_q + \tfrac{1}{2}\gamma B N_\gamma \quad (\text{ B = bearing width }) \quad \ldots\ldots \; 3$$

$$q_{cn} = \left(N_q\right)_{sand} = \left(N_c \left[\tfrac{c}{p}\right] + \frac{\sigma_{total}}{\bar{\sigma}_v}\right)_{clay} \quad \ldots\ldots\ldots\ldots\ldots\ldots \; 4$$

$$f_{sn} = \frac{f_s}{\bar{\sigma}_v} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \; 5$$

$$N_1 = N\, C_n = \frac{N}{[\bar{\sigma}_v]^n} \cong \frac{N}{[\bar{\sigma}_v]^{1/2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \; 6$$

$\bar{\sigma}_v$ = effective vertical stress (tsf)

The exponential n value accounts for the failure envelope curvature and changing dilative characteristics with confining stress increase. If the expotential n value is 0, the measured parameter has no dependence on vertical stress; whereas, if n is 1, there is a linear dependence such as the typical clay strength increase with effective vertical stress (i.e., c/p ratio). The n values have been back-calculated for this study from several large laboratory chamber (i.e., 3-ft-diam) studies and range from 0.58 to 0.86 for the CPT and from 0.42 to 0.56 for the SPT. Dense gravels and sands appear to have the lowest n value with clays having an n value near 1, while medium dense fine sands have an n value near 0.70.

A simple normalization technique for the f_s is shown in Equation 5 for direct comparison to an assumed undrained strength to vertical effective stress ratio (i.e., c/p ratio) because the sleeve friction is a measure of soil. sensitivity effected strength.

CPT DATA PRESENTATION

We now have two normalization parameters, q_{cn} for soil consistency and f_{sn} for soil sensitivity as shown in Figure 1: CPT Soil Characterization chart. The soil characterization lines (i.e., soil-type boundaries) for this chart originated from the CPT Soil Behavior Chart (Ref.1) and were refined based on normalized CPT data. The CPT soil classification is based on mechanical soil properties rather than strictly grain size limits (Ref.1). Calculating the q_{cn} requires the expotential n value, which is estimated by associating it with the soil characterization lines as shown in Figure 1. Calculating the q_{cn} will generally require an iterative solution starting with the lowest n value and progressing until the assumed n value and calculated q_{cn} matches the n value and q_{cn} on a constant f_{sn} line from the chart.

A narrow band was established for normally consolidated nonsensitive packed soil of uniform grain size as shown in Figure 1 and is based on CPT data from sites with known stress/strength conditions. Overconsolidation causes an increasing q_{cn} and f_{sn} at a slope away from a point on this band, while increasing sensitivity falls to the left of this band at a shallow slope as shown by the arrows; these slopes are also dependent on soil type. Note the descriptions in Figure 1 concerning overconsolidation and sensitivity trends.

CPT-TO-SPT CORRELATION

Prediction of the SPT using the CPT is an important step in order to use the large SPT data base of liquefaction correlation and the established SPT liquefaction techniques (Refs.5 and 6). The field SPT N value is the blow count to advance the SPT sampler from 6 to 18 inches below the bottom of the borehole (Ref.7) and is normalized to N_1 using the C_n parameter (ref.4) as shown in Equation 6. The SPT sampler resistance is derived from (1) SPT side friction with the contact area increasing with greater penetration below the bottom of the borehole, (2) SPT end bearing for each hammer blow, and (3) SPT friction inside the cutting shoe. The SPT end bearing will dominate the total resistance in dense sands; whereas, the SPT skin friction will dominate in loose sands through clays. By modeling the stresses on the SPT sampler using CPT measurements, a nonlinear relationship of sampling energy (based on CPT data) to measured SPT blow count can be established with an apparent SPT efficiency lower then the anticipated 55 percent (Ref.8). From the previous section on soil sensitivity, it was described that the average n value for the SPT (i.e., 0.5) is lower than the n value for the CPT (i.e., 0.7), which could indicate lower stress dependence for the SPT technique. This n value effect and the lower apparent calculated SPT efficiency (Ref.8) both indicate a probable combination of soil disturbance at the bottom of the borehole and the open borehole creating a lower than vertical stress at the SPT sampler location especially for the first 6 inches below the bottom of the borehole. The SPT side friction for the first 6 inches below the borehole can account for as much as 50 percent of the total SPT side resistance when the SPT sampler penetrates from 6 to 18 inches below the bottom of the borehole. Therefore, the SPT blow count is a crude index of a complicated process and the C_n factor should only be applied to SPT N value and the C_p factor to the CPT q_c.

A good CPT-to-SPT correlation can only be achieved at a uniform soil site of standard normally consolidated soil using a controlled field SPT procedure. The CPT measurements must be constant over at least the length of the SPT sampler for a proper site confirmation. When confirmation is poor, the SPT will generally be the problem because the SPT procedure is difficult to control while the CPT q_c and f_s can be thought of as components of the SPT resistance. A point-to-point correlation of CPT to SPT is difficult, but the generalized SPT average for normally consolidated sites is valid for developing the CPT liquefaction technique.

The CPT and SPT were normalized using Figure 1 and Equation 6 with a major part of the data base coming from controlled field studies (Ref.9); the resulting trends are summarized in Figure 2 in terms of zones for SPT N_1 equal to 5, 10, 20 and 40. The SPT trends indicate a unique relation of increasing q_{cn} with decreasing f_{sn} for constant SPT contours; this is the same relationship that was described for a possible constant liquefaction potential in the previous section on soil sensitivity. Therefore, the SPT could be a good index of liquefaction assessment except for the field procedure problems. Non-normalized CPT-to-SPT relationships by others (Refs.2 and 9) are also shown in Figure 2 for comparison with the N_1 zones from this study using normalization concepts.

The conventional method for SPT prediction using CPT data is the q_c/N factor, which is the ratio of CPT cone resistance to SPT blow count. For sands, back-calculated q_c/N ranges between 1 and 10 (Ref.2). By properly modeling the incremental stress increase on the SPT sampler based on CPT data (Ref.8), the derived q_c/N for sand ranges between 2 and 6. The q_{cn}/N_1 was used in lieu of q_c/N because the two parameters can be more meaningfully compared if both are properly normalized. The calculated q_{cn}/N_1 for the average of the SPT zones are shown at the top of Figure 2, and reflect that a q_{cn}/N_1 of 4.5 for sand and 4 for silty sand (Ref.5) are conservative but adequate for CPT-to-SPT calculations (Ref.10).

CPT TO CYCLIC LABORATORY DATA AND FIELD OBSERVED LIQUEFACTION

The first requirement for laboratory and CPT comparison is a new means of expressing laboratory cyclic data. This requires normalizing the cyclic laboratory data in the form of an equation and in terms of earthquake level and vertical effective stress. The site liquefaction resistance strength ratio ($\tau/\bar{\sigma}_v$) which depends on the earthquake level and vertical effective stress, is calculated using the CPT-derived normalizied liquefaction resistance strength ratio ($\tau_n/\bar{\sigma}_v$) as shown in Equations 7 through 9:

$$\frac{\tau}{\bar{\sigma}_v} = \left(\frac{\tau_n}{\bar{\sigma}_v}\right)\frac{C_e}{[\bar{\sigma}_v]^f} \cong \left(\frac{\tau_n}{\bar{\sigma}_v}\right)\frac{C_e}{[\bar{\sigma}_v]^{0.3}} \quad \ldots\ldots 7$$

$$C_e = \left(\frac{15}{N}\right)^c \cong \left(\frac{15}{N}\right)^{0.285} \quad \ldots\ldots 8$$

OR

$$= \left(\frac{7.5}{M}\right)^m \cong \left(\frac{7.5}{M}\right)^{1.2} \quad \ldots\ldots 9$$

c, f, m = exponential values (see text below)
$\bar{\sigma}_v$ = vertical effective stress (tsf or Kg/cm^2)
N = design number of earthquake cycles
M = design earthquake magnitude
τ_n = normalized liquefaction resistance strength from CPT or SPT

analysis at an equivalent vertical effective stress of 1 tsf for a 7.5 magnitude earthquake (15 equivalent cycles), and level ground conditions.

The f value from Equation 7 is required to account for the observed curvature of the laboratory determined liquefaction resistance strength to vertical effective stress failure envelope and generally range from 0.05 to 0.4 based on back-calculated laboratory data with a typical equal to 0.3. The m value for Equation 8 and the c value for Equation 9 are required to quantify the rate of liquefaction resistance strength loss with increasing cycles in terms of earthquake magnitude or earthquake equivalent number of cycles. The state-of-the-art method for relating the earthquake magnitude and number of cycles to the liquefaction resistance (Ref.11) when back-calculated produces an m equal to 1.2 and c equal to 0.285.

Finally, using the normalization concepts from Figure 1 and Equation 7, CPT to cyclic laboratory triaxial and simple shear data from several sites are shown in Figure 3. Correlations of cyclic laboratory data and CPT data were only attempted when the CPT q_c and f_s were nearly constant over at least the length of the Shelby tube sampler. The shape and size of the semicircles in Figure 3 reflect the CPT variation over the depth of the soil sampler used for laboratory testing.

One-to-one correlation of field-observed liquefaction to in-situ testing data appears to be the ultimate match, but it should only be attempted in uniform deposits with constant dynamic strength and at the fringe of the liquefied zone. Unfortunately uniform deposits do not occur in nature. When there is surface expression of liquefaction in a nonuniform deposit, the major difficultly is determining the layers which did liquefy.

CPT LIQUEFACTION TECHNIQUE

Agreement was found between the CPT-laboratory liquefaction data (Figure 3), the Tokimatsu-Yoshimi SPT liquefaction technique (Ref.6) and the CPT-SPT trends (Figure 2) for clean sand having SPT N_1 values below 15. Therefore the average CPT-SPT N_1 trend and the Seed SPT liquefaction technique (Ref.5) should at least predict conservative liquefaction resistance strength for SPT N_1 values below 15. The next step to is employ the observation that the liquefaction resistance of clean sand (i.e., $D_{50} > 0.25$ mm) is essentially equal to dirty sand (i.e., $D_{50} = 0.15$ mm) plus 7.5 (Refs.5 and 6). Therefore, estimates were made for the soil characteristic sublines having an equivalent uniform grain size with a D_{50} of 0.15 and 0.25 mm.

Normalized liquefaction resistance strength ratio contours are shown in Figure 4, and reflect general agreement between the SPT liquefaction technique and the laboratory cyclic data in the overlap silty sand zone. A narrative description of the probable liquefaction behavior of the various zones are also shown in figure 4. China CPT to field observed liquefaction data (ref.12) was not used to develop these contours, but they were used as validation.

CONCLUSION

A new rational method for evaluating liquefaction potential using the CPT has been presented. It requires CPT data normalization using the iterative exponential n value as shown in Figure 1 to predict a normalized liquefaction resistance strength ratio (i.e., cyclic stress ratio) from Figure 4 which is then converted to a site liquefaction resistance strength with Equation 7. The CPT should be used first as a stratigraphy tool and second as a means of liquefaction assessment. Where marginal material exists as suggested by the CPT liquefaction technique (i.e., factor of safety < 1.3), soil samples should be retrieved based on CPT stratigraphy for laboratory confirmation tests.

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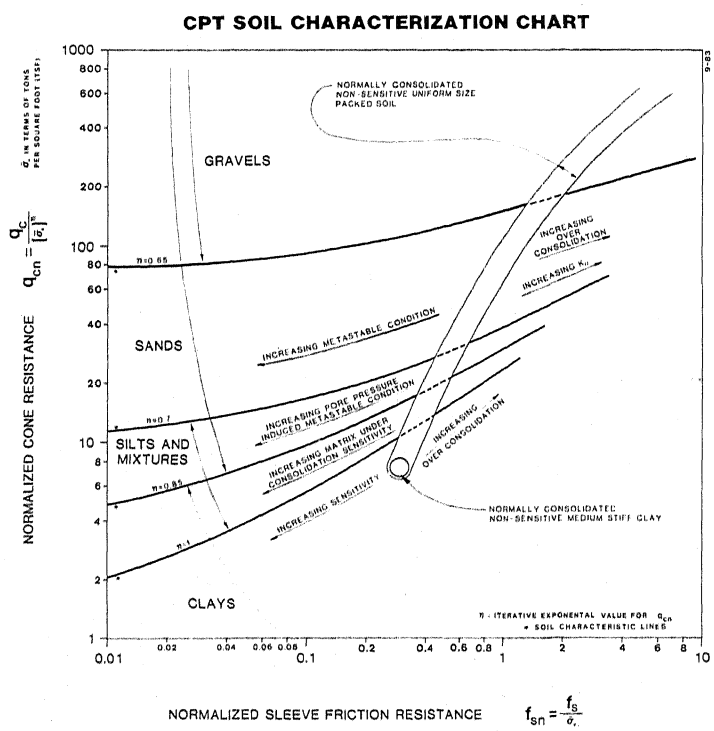

Figure 1. Cone Penetrometer Test (CPT) Soil Charaterization chart

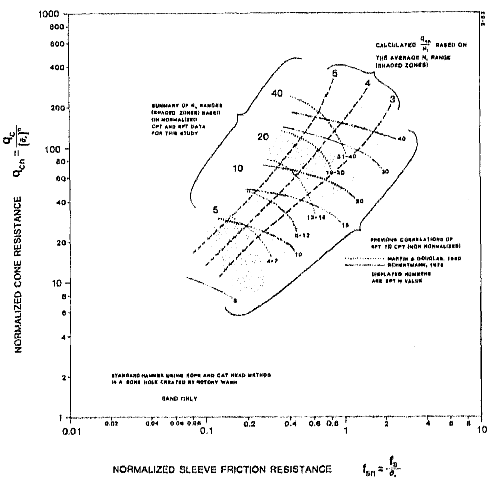

Figure 2. Comparison of SPT N_1 to the normalized CPT relationship

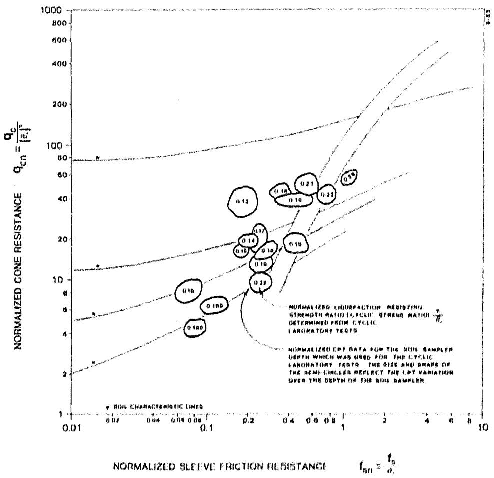

Figure 3. Comparison of normalized cyclic laboratory data to the normalized CPT relationship

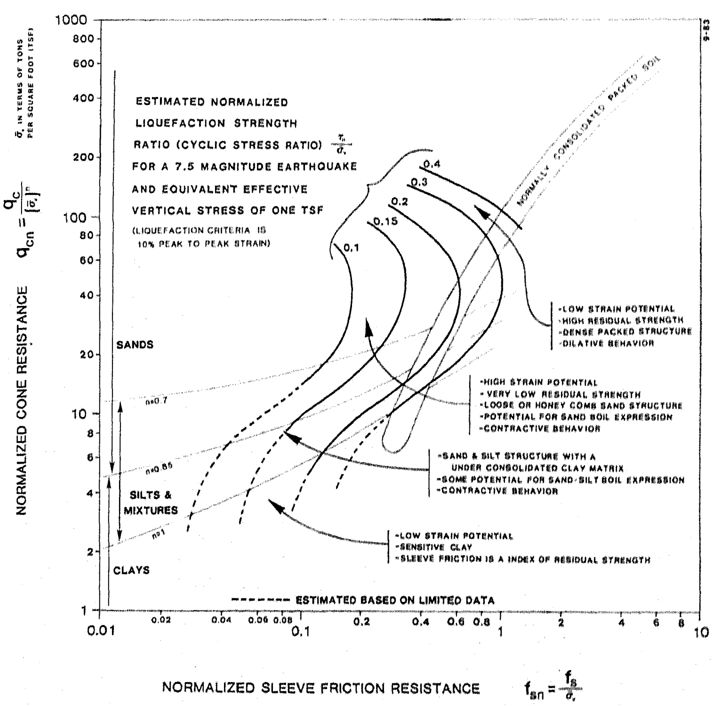

Figure 4. Correlation between estimated normalized liquefaction resistance strength ratio (i.e., cyclic stress ratio) and the normalized Cone Penetrometer (CPT) relationship