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Literature Review on Liquefaction Screening using CPT

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Abstract - *Assessing liquefaction screening and potential for clean sands and silty sands remains a difficult problem: how the fines content affects the cyclic resistance ratio (CRR), consolidation characteristics, and cone resistance in CPT still not clearly understood. To assess liquefaction potential, liquefaction screening charts depicting a correlation between cyclic resistance ratio (CRR) and normalized cone resistance (q_{c1N}) has been evolved based upon past earthquake case histories and laboratory study. Whether a site would be liquefiable or non-liquefiable can be crudely evaluated by the demarcation line indicating the relationship between CRR and q_{c1N}. It has been observed that the demarcation line shifting towards left with the increases of silty content: sands and silty sands with the same cone resistance can have different liquefaction resistance. In field observation, the difficulty in assessing the in-situ condition during earthquake leads the researchers to assess the liquefaction potential of corresponding soil in laboratory study. In this paper, how the silt content alters the intergrain contact density, affect the cyclic resistance, consolidation characteristics have been presented. The reduction in cone resistance, shifting the CRR-q_{c1N} curves towards left as silt content increases, can be clearly understood by coefficient of consolidation rather than gross silt content. In laboratory CPT, the effect of silt content on normalized cone resistance (q_{c1N}) has been observed through intergrain contact density and consolidation characteristics. Normalized penetration rate, T_o (=v*dc/cv, where v is the cone penetration rate, dc is the cone diameter of the cone and cv is the coefficient of consolidation) has been introduced into the CRR-q_{c1N} charts and compared with the current liquefaction screening charts based on CRR-q_{c1N}-FC, where FC is the percent silt content.*

Key Words: **Liquefaction, Cone resistance, Silt content, density, consolidation, permeability**

1. INTRODUCTION

The term liquefaction has been used in concurrence with a variety of phenomena that includes soil deformations caused by disturbance from monotonic, transient, or repeated loading of saturated cohesionless soils under partially undrained or undrained conditions. Developing excess pore water pressure and dissipation of the pressure is the main factor in all liquefaction phenomena. As shown in the figure below, when a soil is experiencing effective stress large enough to cause slip among grain contacts, the contacts between soil particles become instable and transfer the forces carried by contacts to water hence develop the pore water pressure.

1.1 Liquefaction phenomena

Liquefaction phenomena could be divided into two main categories: flow liquefaction and cyclic softening. Flow liquefaction can happen when the shear stress necessary for static equilibrium of a soil mass is greater than the shear strength of the soil in its liquefied state [6],[7]. In contrast to flow liquefaction, cyclic mobility occurs when the static shear stress is less than the shear strength of the liquefied soil. There have been many studies focused on identifying the factors and mechanisms causing liquefaction [5], [13], [17]. Size, shape, and gradation spectrum of soil particles, initial relative density, fines content, stress level, drainage characteristics, previous strain history, vibration characteristics, period of loading, and tapped air are the main factors affect the liquefaction process

2. Literature review

Liquefaction screening techniques, empirical relation between liquefaction resistance and field data evolved based upon knowledge from laboratory research conducted on clean sands and observed field performance during past earthquakes [24], are to measure the liquefaction potential of a site. To evaluate the soil in liquefaction resistance capacity, researcher began to study previous case history of sites: liquefied and non-liquefied during earthquake. The soil resistance is measured in the study site by the available field test like Standard Penetration Test (SPT), Cone Penetration Test (CPT) and Shear Wave velocity (Vs). Fig-1 presents the liquefaction resistance, cyclic stress ratio (CSR) and soil resistance $(N_1)_{60}$ of different sites. Demarcation line have been drawn between liquefied and non-liquefied sites. Current liquefaction screening of clean sands and silty sands from in-situ tests results, standard penetration test (SPT), cone penetration test (CPT) and shear wave velocity, have been correlated to cyclic resistance ratio (CRR) for different fines content [1], [11], [10], [14], [16]. Field observation of liquefaction screening provides the clean sand base curve and suggest an extrapolation to account fines content in soils. However, how the fines affect the liquefaction potential assessment is not clearly understood which leads to researcher to explore silt-phenomenon in the liquefaction screening chart.

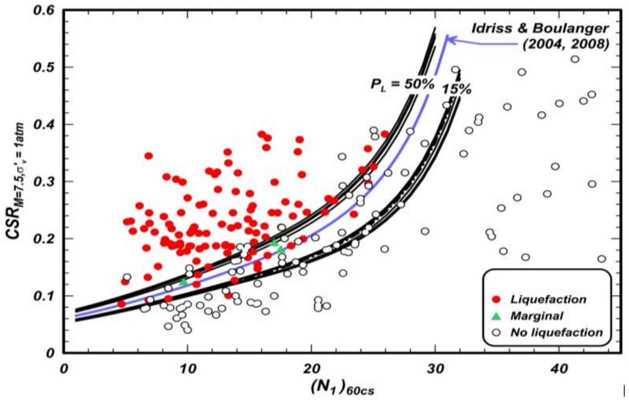

Fig-1: Cyclic Stress Ratio vs Soil Resistance [4], [10]

It has been observed from the Fig-1, at the same penetration resistance, $(N_1)_{60}$, soil has different liquefaction resistance which leads the researcher to study the factors causing liquefaction other than density. The ability of continuous recording of soil profile, cone penetration test (CPT) has been utilized to brand the liquefaction potential of soils by relating the normalized cone tip resistance-q_{c1N} to the cyclic resistance ratio (CRR) – Fig-2.

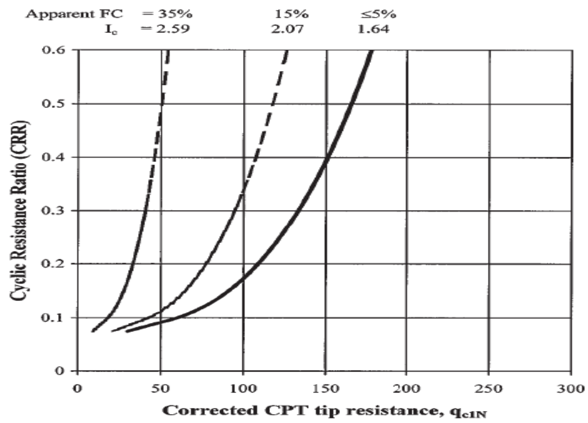

Fig-2: Cyclic Resistance Ratio (CRR) vs CPT tip resistance [11]

2.1. Liquefaction screening for sands and silty sands using CPT

SPT, CPT and shear wave velocity are the means of current approaches to evaluate potential liquefaction. Among, CPT has gained attention to the researcher because of its continuous profiling of soil strata without missing thin layers and repeatability of the test. [10], [14] converted SPT based potential liquefaction assessment data to equivalent CPT based assessment to evaluate the liquefaction potential as presented in Fig-3.

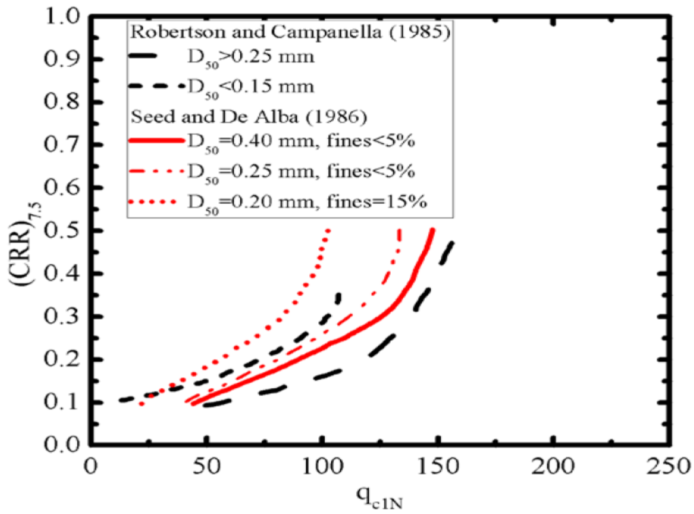

Fig-3: Liquefaction potential evaluation charts using CPT [10], [14]

It is observed in Fig.-3 that coarser soil has lower liquefaction resistance than the finer one. And also showed that sands with fines has more resistance against liquefaction at the same resistance than the clean sands [14].

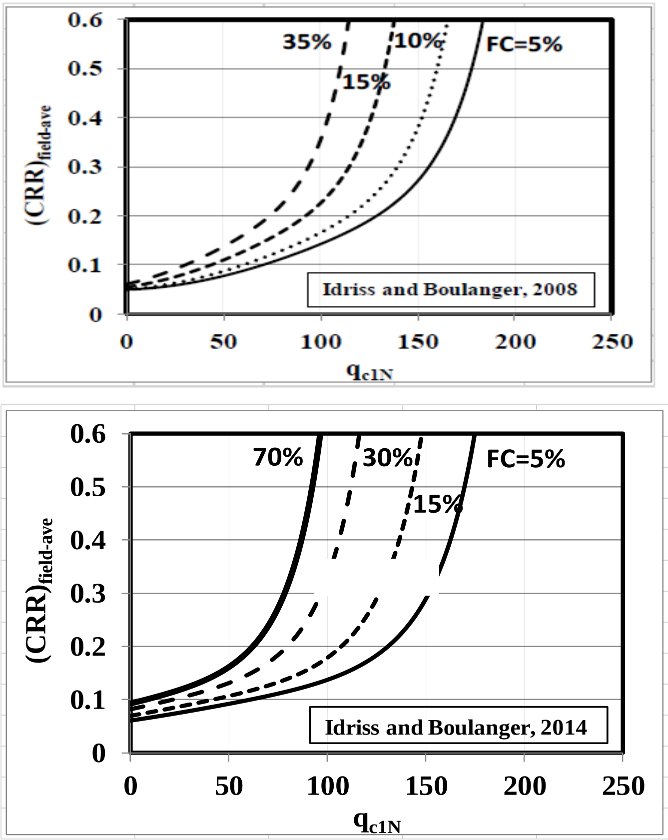

Fig-4: Liquefaction evaluation charts using CPT [4]

Fig-4. presents the correlation between cone resistance (q_{c1N}) and CRR at earthquake magnitude 7.5. It is noticed that the (CRR)7.5-q_{c1N} curves shifting towards left as silts content increases indicating that the clean sand has less liquefaction resistance than the sands with silt. It is also noticed that the liquefaction demarcation line evolving through experience of researcher and practioners. It is also can be concluded from Fig-4 that the silts has little prone to liquefaction as the 100% curve would be almost vertical: Implied from the trend of shifting the curve to the left as fines content increases. And the normalized cone resistance (q_{c1N}) is calculated as:

$$q_{c1N} = \frac{(q_{c1})}{(P_a)} = C_Q * \frac{(q_c)}{(P_a)} \qquad \text{(Eqn. 1)}$$

Where q_c is the field cone resistance and Pa is the atmospheric pressure.

And C_Q is the overburden correction factor defined by [16],

$$C_Q = \frac{1.8}{0.8 + \left(\frac{\sigma'_{vo}}{P_{at}}\right)} \qquad \text{(Eqn. 2)}$$

Where σ'vo is the effective overburden pressure and Pat is the atmospheric pressure.

Or By [24]

$$C_Q = \left(\frac{P_{at}}{\sigma'_{vo}}\right)^n \leq 1.7 \qquad \text{(Eqn. 3)}$$

Where stress exponent (n) varies from around 0.5 in sands to 1.0 in clays. For sands and silty sands 0.5 considered for the n value.

2.2 Liquefaction Screening Based upon Laboratory Study

To understand the characteristics of liquefaction in silty sands, researchers began to investigate the effect of fines on liquefaction resistance [22]. It is reported that the mechanical behaviors of clean sand and silty sand are very similar at a given 'equivalent' inter-granular relative density $(D_{rc})_{eq}$ [23]. The effect of fines content is reflected by permeability (k) and coefficient of consolidation (C_v) in saturated sand-silt mixture [15], [21]. Along with the soil properties (k, C_v), non-soil parameters such as penetration rate of the cone, and diameter of the cone have influence in the reduction of cone resistance in saturated silty sands. Normalized penetration rate T_0 (T_0=v*d_c/c_v, where v is the penetration velocity, d_c is the diameter of cone) incorporated the three factors causing the reduction in saturated test [9]. Huang (2014) designed a new chamber testing, capable of simulating the condition of constant vertical stress and zero lateral strain (ideally)-BC3. Conduct CPTs using Ottawa F55 saturated sand-silt mix at different fines contents (0,15 and 25%) to examine the effect of fines on the cone resistance at different fines content (0, 15 and 25%) [3]. In continuation of Huang (2014) study, Sivaratnarajah (2016) conducted several test at dry and saturated condition on sand-silt mixture to examine the effect of T_o on the cone resistance and justify the findings of Huang 2014 study [12].

2.2.1 Contact Density Index-A Framework to Intergranular Void Ratio

As stated earlier that sands with silts has different characteristic than the clean sands. At the same global void ratio (e) silty sand have different mechanical properties compared with the clean sands because global void ratio indicates only the voids in the sample but does not give how fines oriented in the voids of the coarser grains and how it affects the force chain in the soil matrix. Earlier study showed that the global void is a poor index of soil mixtures. This problem leads researcher to develop the equivalent void ratio concept [17], [23] which proposes that the mechanical properties such as steady state strength, liquefaction resistance (CRR), stress-strain characteristics are expected to be same for both clean sand and silty sand at the same contact density index. To address this effect, [20], [23] developed a soil matrix classification system.

2.2.2 Normalized Penetration Rate

As stated earlier, the evaluation process of liquefaction potential based upon q_{C1N} – CRR curve does not give any clear idea how fines affect the cone resistance. The essence of fines in the liquefaction resistance leads further research on the effect of fines content in the difference in cone resistance for silty sands and other soil mixtures. The difference in cone resistance results from the various drainage conditions when the cone advances into different soils: for example, the clean sand undergoes drained condition as it has high permeability and the excess pore water pressure dissipates fast thus no significant reduction. As the penetration rate (v) and the diameter of the cone (d_c) also contribute to the cone resistance a non-dimensional penetration rate T_o [9] is introduce to show the combined effect of degree of consolidation during penetration as below:

$$T_o = \frac{v * d_c}{C_v} \qquad \text{(Eqn. 4)}$$

Where,

T_o= Normalized penetration rate

V= velocity of cone

d_c = diameter of cone

c_v = coefficient of consolidation

2.2.3 Cyclic Stress Ratio (CSR)

According to [13], the cyclic stress ratio, CSR, as the average cyclic shear stress, τ_{av}, developed on the horizontal surface of soil layers due to vertically propagating shear waves normalized by the initial vertical effective stress, σ_v', to include the increase in shear strength due to increase in effective stress. By accordingly weighting the individual stress cycles based on laboratory test data, it has been found that a rational amplitude to use for the "average" or equivalent uniform stress, τav, is about 65% of the maximum shear stress.

$$CSR = \frac{\tau_{av}}{\sigma_v'} = 0.65\left(\frac{a_{max}}{g}\right)\left(\frac{\sigma_v}{\sigma_v'}\right) r_d \text{....... (Eqn.5)}$$

where

a_{max} = maximum horizontal ground surface acceleration (g)

g = gravitational acceleration

σ_v = total overburden pressure at depth z

σ_v' = effective overburden pressure at depth z

r_d = stress reduction factor

The stress reduction factor, r_d, is used to determine the maximum shear stress at different depths in the soil. As shown in the Fig-5 r_d, values generally range from 1 at the ground surface to lower values at larger depths.

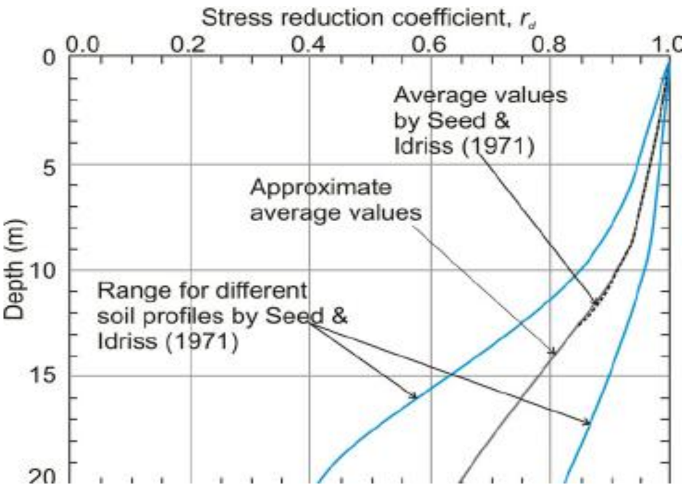

Fig-5: Reduction factor to estimate the variation of cyclic shear stress with depth below level or gently sloping ground surface [24]

2.2.4 Cyclic Resistance Ratio (CRR)

The cyclic resistance ratio (CRR) represents the maximum cyclic stress ratio (CSR) at which a given soil can resist the liquefaction. In summary, CRR is the capacity of the soil against liquefaction and CSR is the field stress induced by shear loading.

Factor of safety against liquefaction,

$$FS_L = (MSF^* \, CRR_{7.5}) / (CSR) \text{ .... (Eqn. 6)}$$

Where MSF is the magnitude scaling factor, CRR7.5 is the cyclic resistance ration at earthquake magnitude 7.5.

Fig-6 shows comparison between in-situ evolved CRR-q_{c1N} and laboratory calculated CRR from laboratory CPT. The effect normalized penetration rate, T_o on the CRR and qc1N has been observed from the figure. The prime limitation for this comparison of CRR from laboratory study and in-situ test is the equivalent conversion factors like earth pressure coefficient, K_o. The earth pressure coefficient is dependent upon the stress history, soil age, and relative density of soil. It is difficult to determine the in-situ earth pressure coefficient hence the converted CRR might not give exact in-situ field condition. The cyclic resistance ratio from triaxial test, (CRR)tx is presumed to be the function of relative density only, which is questionable because the liquefaction resistance is depend on many other factors, like silt content, drainage condition.

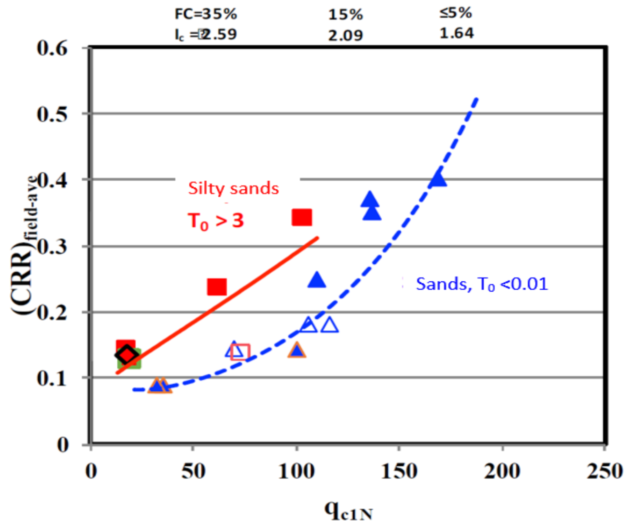

Fig-6: Comparison of laboratory (CRR) to field computed CRR [12]

3. CONCLUSIONS

The current liquefaction screening technique by using CPT primarily rely on knowledge from extensive laboratory study conducted on clean sands and extrapolation of observed field performance of past earthquake. It has been observed that the demarcation line delineating liquefiable and non-liquefiable soil shifting towards left as silt content increases in liquefaction screening charts: liquefaction resistance (CRR) increases with the increases of silt content in soil at the same cone resistance (q_{c1N}). There is no clear consensus how the silt content affects the liquefaction resistance and penetration resistance which leads the researcher to investigate the silt-phenomena.

Based upon the literature review, the effect of silt content in the liquefaction screening technique can be summarized through the following manners:

i) Silt particles in silty sands partially contribute to cyclic resistance ratio CRR and cone resistance qc1N in contrast to its full contribution to the density: liquefaction resistance and cone resistance of silty sands may not be as high as that of clean sands.

ii) Silts content, reduces the pore size and the permeability, inhibits the dissipation of developed excess pore pressure around the cone which leads a reduction in cone resistance.

iii) Normalized cone penetration rate, T_o have been introduced gross fines content in liquefaction screening chart to incorporate coefficient of consolidation and non-soil property like velocity of cone penetration and diameter of the cone in the liquefaction screening chart. It is highly likely that there exist a relationship between CRR-q_{c1N}-T_o.

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