Evaluation of Effectiveness of Ground Improvement for Liquefiable Deposits by Vibro-compaction from CPTU Data

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ABSTRACT: Vibro-compaction is a new method for liquefaction ground treatment in China. To evaluate the geotechnical characterization of liquefaction foundation improved by the vibro-compaction, a series of field tests including piezocone penetration tests (CPTU) were performed before and after ground treatment in the construction activity of highway in Suqian, China. According to critical state theory, an approach to evaluate the improvement level in liquefiable soils treated by vibro-compaction was developed on the basis of the state parameter (Ψ) of sand soils. Some meaningful results were obtained through analyzing relative density (Dr), state parameter (Ψ), factor of safety (Fs) and probability of liquefaction (PL) before and after vibro-compaction. The high consistency of liquefaction evaluations indicate that the vibro-compaction could be used for ground improvement needed to mitigate liquefaction.

KEYWORDS: ground improvement, vibro-compaction, state parameter, liquefaction, piezocone penetration test

1. INTRODUCTION
Soil liquefaction is a major concern of highway foundation failure at sites of loose silt or sand, which can be reduced by various ground improvement methods. Such as dynamic compaction, vibroflotation, vibro-stone columns etc. However, these methods often require fillers, high cost, high-impact vibration on the surrounding environment, the processing depth is limited, therefore, and it is often limited in engineering applications. Vibro-compaction is a soil densification method of ground improvement that used commonly in the worldwide. The method has been developed since 20th century, and has been playing an important role in foundation improvement all the time. Among these methods of ground improvement, vibro-compaction is a proven ground improvement method for liquefaction mitigation of loose granular soils or sands, improve the soil properties and the overall seismic performance. Massarsch presented the features of different, purpose-built types of compaction probes and special equipment for deep vibro-compaction and studied the most important factors governing the compaction process based on CPTU test. For those associated liquefaction fields, how to evaluate the effectiveness of ground improvement using vibro-compaction method is still a very important and meaning task.

Vibro-compaction is a way that makes soil and loose soil particles are repacked into a more compact state. However, no detailed procedures are available to determine the densification achievable during vibro-compaction. The current state of practice depends mainly on previous experience or field test programs to determine the applicability of the technique at a given site. The critical state (or steady state) is a useful concept for evaluating the liquefaction
potential of a soil deposit. In the critical state soil mechanics, the stress state of the specimen reaches critical state line when the mean stress $p$, deviatoric stress $q$ and volume are unchanged. Nevertheless, few studies have been conducted on the critical state of sand or silt.

The objectives of this study are to evaluate foundation liquefaction improved by the vibro-compaction method with a cross-shaped vibration wing. Based on CPTU field tests. An approach to evaluate the improvement level in liquefiable soils treated by vibro-compaction was developed based on state parameter ($\Psi$) of sandy soils. Some parameters of relative density ($D_r$), state parameter ($\Psi$), factor of safety ($F_s$) and probability of liquefaction ($P_L$) before and after vibro-compaction were analysed for liquefaction evaluations, which prove that the vibro-compaction could be used for ground improvement needed to mitigate liquefaction.

2. SELF-DEVELOPMENT VIBRO-COMPACTION METHOD

The equipment consists in general of three parts: a 7 tons vibrator with power pack, a cross-shaped vibro-wing and a 50-ton crawler crane. The vibrator is attached to the top of a 0.6m wide and 15m long probe. The probe is provided with circular openings of 0.1m in diameter and 0.8m apart aiming to reduce probe impedance and provide better contact with soil. The cross-shaped vibration wing is a cylindrical probe with two perpendicular steel plates, as shown in Fig. 1. The vertical oscillation is generated by the eccentric weights situated at the top of the cylinder. The range of vibration frequency varying from 0 to 20 Hz. The vibro-wing is lowered to the necessary treatment depth at a rate of 2.0 m/min, and raised at a rate of 1.2 m/min.

3. SITE DESCRIPTIONS AND PIEZOCONE PENETRATION TEST

The test site situates in a construction activity of highway in Suqian, Jiangsu Province, China. The basic seismic intensity in the region is 8 degrees and the maximum ground acceleration is 0.25 g. The upper part of the formation is mainly silt, which is easy to produce water and soil liquefaction. The main geological structure is an active fault zones that seismic activity. The CPTU is a popular repeatable situ test than other situ tests, which can produce a continuous profile of soil and save much time. The main parts of a piezocone penetrometer are depicted in Fig.2 according to ASTM D5778-12. A digital penetrometer is hydraulically pushed at a penetration rate of 2 cm/s to collect continuous readings of cone tip resistance ($q_t$), sleeve friction ($f_s$), and pore water pressures ($u_2$).
4. DEFINITIONS OF CRITICAL STATE AND STATE PARAMETER

In order to evaluate effectiveness of ground improvement by Vibro-compaction, some content of state parameter rare firstly introduced. Based on critical state soil mechanics, an important concept that the state parameter $\Psi$ is developed by Been and Jefferies. The definition of state parameter $\Psi$ is the difference between the void ratio of the current state and that of a critical state at the same mean effective stress (Fig 3). That is to say, the state parameter $\Psi$ is determined by the void ratio ($e$) and effective stress level ($p'$) of a sand relative to a critical state line. When $\Psi$ is positive or sand with a void ratio above CSL behaves contractively under loading. When $\Psi$ negative or a void ratio below CSL is tend to be a dilative behavior.

![Diagram of state parameter definition](image)

**Figure 3. Definition of state parameter.**

The state parameter presented in Fig.3is determined as follows:

$$\Psi = e_0 - e_{cs}$$  \hspace{1cm} (1)

Where $\Psi$ is state parameter, $e_0$ and $e_{cs}$ are the initial and critical void ratios, respectively.

The state parameter $\Psi$ combines the influence of relative density and stress level on sand, which are the two most influential state variables on soil behavior. Been et al. (1985) also suggested a correlation between the normalized cone resistance and state parameter as follows:

$$Q_p = \frac{q_e - p}{p'} = k \exp(-m\Psi)$$  \hspace{1cm} (2)

Where $Q_p$ is a normalized cone resistance, $p$ and $p'$ are the mean total and effective stresses, respectively, and $k$ and $m$ are functions of compressibility. The values of $k$ and $m$ both are related to $\lambda$, which is the slope of the critical state line.

5. ANALYSIS OF TEST RESULTS

5.1 Basic parameters of CPTU test

A representative CPTU test result (ground unimproved and improved) are given
in Fig. 4. As shown, before reinforcement, the cone tip resistance is generally small in silt and silty sand. Aggregation and dissipation of excess pore water pressure are not observed in the CPTU profiles, indicating that the soils are in a relatively loose state and there is the possibility of liquefaction. After reinforcement, it can be observed that the cone tip resistance becomes larger in silt and silty sand. The strength increased significantly in liquefied soil layer of silt ② and silty sand ③. The strength of the soil layer ④ that outside of reinforcement depth has been also improved and the effect of reinforcement depth exceed the scope of design. The underground water table in the test site fluctuates from 3.6 m.

Figure 4. Comparison results of CPTU.

5.2 Relative Density
For cohesionless soils, $D_r$ represents the degree of compactness with regard to both the loosest and densest states. In this study, $D_r$ is determined by the method proposed by Jamiolkowski. Which incorporates a correction factor, considering the influence of the vertical effective stress $\sigma_{\phi}'$, can be expressed by:

$$D_r = 100 \left[ 0.268 \ln \frac{q_t}{\sigma_{\phi}'} - 0.675 \right]$$

Where $q_t$ is piezocone tip resistance, $\sigma_{\phi}'$ is the vertical effective stress, kPa; and $\sigma_{\text{atm}}$ is normal atmospheric pressure.

The classification criterion of sand soil compactness according to relative density $D_r$ is shown in Table 1. Based on the CPTU data of liquefaction-susceptible field, the changes of $D_r$ before and after treatment can be estimated. The results are given in Table 2. It can be observed that the relative density increases with the depth and the surface soil is in middle dense state, but the layer of silt ②, silty sand ③ and sand ④ are in the loose ~ middle dense state. After reinforcement, the relative density of the liquefied soil layer ② and ③ have increased by 67%, 154%, respectively. The relative density of the soil layer ④ that outside of reinforcement depth has been also improved. The results are in accordance with the previous analysis.

Table 1. Partition density index according to relative density.

<table>
<thead>
<tr>
<th>Density</th>
<th>Dense</th>
<th>Middle dense</th>
<th>Loose</th>
</tr>
</thead>
<tbody>
<tr>
<td>$D_r$</td>
<td>$D_r &gt; 2/3$</td>
<td>$2/3 \geq D_r &gt; 1/3$</td>
<td>$D_r \leq 1/3$</td>
</tr>
</tbody>
</table>
Table 2. Evaluation of relative density.

<table>
<thead>
<tr>
<th>Soil type</th>
<th>Thickness</th>
<th>Test results of $D_r$</th>
<th>Increase rate</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Before reinforcement</td>
<td>After reinforcement</td>
</tr>
<tr>
<td>①Fill</td>
<td>0-1.5</td>
<td>49.5</td>
<td>24.1</td>
</tr>
<tr>
<td>②Silt</td>
<td>1.5-5.3</td>
<td>24.5</td>
<td>40.9</td>
</tr>
<tr>
<td>③Siltysand</td>
<td>5.3~14.7</td>
<td>22.8</td>
<td>57.9</td>
</tr>
<tr>
<td>④Sand</td>
<td>14.7~20</td>
<td>25.4</td>
<td>45.2</td>
</tr>
</tbody>
</table>

5.3 State Parameter

State parameter $\Psi$ that is calculated by Jefferies and been [10] are used. A representative test results are given in Fig.5. Before the ground reinforcement, the state parameters are negative or positive but close to zero in layers of silt, silt and sand. When the state parameter value is greater than zero (i.e., $\Psi > 0$), the possibility of liquefaction is high in silt or sand. After reinforcement, the state parameters are negative and away from zero, which can be shown that there is little possibility of liquefaction. It also proves effectiveness of the method of vibro-compaction to treat liquefied foundation.

![Figure 5. Comparison results of the parameters before and after reinforcement.](image)

5.4 The Relationship of State Parameter and Relative Density

This single parameter embodies quantitative information on both the current state of the soil element and the volume change potential of the soil itself. Therefore, the relationship of state parameter and relative density has developed as shown in Fig.6. It should be noted that when the state parameter decreases, the relative density of sand increases and the soil may be prone to unliquefied. On the contrary, when the state parameter increases, the relative density of sand decreases and the soil may be prone to be liquefied. We can judge the density degree of the soil according to the state parameter. It can be seen that the related relationship between the state parameter and the relative compactness is:

$$\Psi = 0.025-0.33D_r, R^2 = 0.79$$

(4)
The correlation between the state parameter and the relative density is good, so the state parameter can replace relative density to both express the soil condition and apply to liquefaction evaluations. Just as Jefferies and been established one semi-theoretical CRR-$\psi$ correlation essentially based on a large of field case history data on liquefaction. Some researchers have also used state parameter to evaluate the liquefaction potential.

### 5.5 Liquefaction Condition

#### Evaluating Factor of Safety

The Factor of Safety ($F_S$) is defined as $F_S = CRR / CSR$, which can be calculated by the formula to calculate the cycle resistance ratio ($CRR$) and the cycle stress ratio ($CSR$). Based on the measured data of CPTU test, the values of $F_S$ are estimated before and after reinforcement (Table 3). Table 3 indicate the values of $F_S$ of liquefiable soil layer ② and ③ have increased significantly, at least greater than one time. It is proved that the site is safe after reinforcement.

<table>
<thead>
<tr>
<th>Soil type</th>
<th>Thickness</th>
<th>Test results of $F_S$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Before reinforcement</td>
</tr>
<tr>
<td>①Fill</td>
<td>0-1.5</td>
<td>2.60</td>
</tr>
<tr>
<td>②Silt</td>
<td>1.5-5.3</td>
<td>0.91</td>
</tr>
<tr>
<td>③Silty sand</td>
<td>5.3-14.7</td>
<td>0.96</td>
</tr>
<tr>
<td>④Sand</td>
<td>14.7-20</td>
<td>1.41</td>
</tr>
</tbody>
</table>

#### Evaluating of the Probability of Liquefactions

Chinese seismic design code of liquefaction standard is not based on reliability. Different intensities may have different levels of risk. The current standard for expression is unsuitable for design decision analysis of important engineering liquefaction risk. Juang and Chen (2000) [13] studied the correlation between Factor of Safety ($F_s$) and Probability of Liquefactions ($P_L$) based on a large number of liquefaction site CPT test. The classification is presented in Table 4, the result of $F_s$ and $P_L$ are shown in Fig 5 and the results of estimated the values of $P_L$ before and after reinforcement in Table 5.
### Table 4. The classification of liquefaction potential.

<table>
<thead>
<tr>
<th>Classification</th>
<th>Probability of liquefactions</th>
<th>Liquefaction potential</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0 ≤ ( P_L &lt; 0.15 )</td>
<td>Non liquefaction</td>
</tr>
<tr>
<td>2</td>
<td>0.15 ≤ ( P_L &lt; 0.35 )</td>
<td>unlikely</td>
</tr>
<tr>
<td>3</td>
<td>0.35 ≤ ( P_L &lt; 0.65 )</td>
<td>liquefaction / non- liquefaction is equally likely</td>
</tr>
<tr>
<td>4</td>
<td>0.65 ≤ ( P_L &lt; 0.85 )</td>
<td>Very likely liquefied</td>
</tr>
<tr>
<td>5</td>
<td>0.85 ≤ ( P_L &lt; 1 )</td>
<td>liquefaction</td>
</tr>
</tbody>
</table>

### Table 5. Probability of liquefactions before and after reinforcement.

<table>
<thead>
<tr>
<th>Soil type</th>
<th>Thickness</th>
<th>Test results of ( P_L )</th>
<th>Decrease amplitude</th>
</tr>
</thead>
<tbody>
<tr>
<td>① Fill</td>
<td>0-1.5</td>
<td>Before reinforcement: 0.18 After reinforcement: 0.63</td>
<td>———</td>
</tr>
<tr>
<td>② Silt</td>
<td>1.5-5.3</td>
<td>0.58 Before reinforcement: 0.33 After reinforcement: 0.07 Decrease amplitude: 43.1%</td>
<td></td>
</tr>
<tr>
<td>③ Silty sand</td>
<td>5.3~14.7</td>
<td>0.55 Before reinforcement: 0.07 After reinforcement: 0.15 Decrease amplitude: 87.3%</td>
<td></td>
</tr>
<tr>
<td>④ Sand</td>
<td>14.7~20</td>
<td>0.26 Before reinforcement: 0.15 After reinforcement: 0.15 Decrease amplitude: 42.3%</td>
<td></td>
</tr>
</tbody>
</table>

Table 5 also shows that after reinforcement liquefiable soil layer ②, ③, and ④ have been improved. The soil layer ④ that outside of reinforcement depth has also improved. The values of \( P_L \) change from 0.26 to 0.15, which decrease 42.31%. This indicates that the soil layer ④ changes from very likely liquefied state to very likely liquefied state. The results obtained are consistent with the previous results that can achieve the purpose to eliminate liquefaction by vibro-compaction method.

As can be seen from the Fig 7(a) and Table 3, the values of \( F_s \) of silt ② and silty sand ③ layers are less than 1. After reinforcement, the values of \( F_s \) are largely more than 1 and eliminate liquefaction. Besides, probability of liquefaction have sharply decreased, which also proved that vibro-compaction can succeed in dealing with these liquefaction foundations.

![Figure 7. Comparison liquefaction condition before and after reinforcement.](image)
6. CONCLUSIONS
The state parameter is widely accepted to represent the soil behavior encompassing both density and stress effects. The required level of ground improvement is supposed to be obtained the state of the sandy soil and evaluation of liquefaction potential of saturated soils based on the state parameter. The high consistency of relative density, state parameter, factor of safety ($F_s$) and probability of liquefacions ($P_L$) indicate that the vibro-compaction could be used for ground improvement needed to mitigate liquefaction.

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