ABSTRACT

The cyclic resistance, shear wave velocity and post-liquefaction behavior of saturated Firoozkooh sand with different percentages of non-plastic silt are evaluated. Cyclic triaxial and resonant column tests performed on reconstituted samples prepared in laboratory utilizing undercompaction method. The data obtained from this study along with other existing data were transferred to the field and compared with the field performance curves based on shear wave velocity proposed by Andrus and Stokoe-2000. Then, to observe post-liquefaction behavior of the mixtures, volume change and pore pressure dissipation were measured. Tests results exhibit a clear trend among cyclic resistance, shear wave velocity and post-liquefaction behavior of specimens. In addition, the laboratory results indicated that using the existing field-based correlations may overestimate the cyclic resistance of the Firoozkooh sand-silt mixtures when silt content is 60%. For clean sand and the specimens with up to 30% fines, results of this study are fairly consistent with Andrus and Stokoe correlations. Increasing fines content would increase the final post-liquefaction volumetric strain.

1. Introduction

Field test results such as the N value from standard penetration test (SPT), cone tip resistance \( q_c \) from cone penetration test (CPT) or shear wave velocity \( V_s \) have been widely used to assess the liquefaction potential for sand under framework of simplified procedures [1]. The simplified procedure uses an empirical correlation between the cyclic resistance ratios (CRR) and the field test results to determine if the soil is potentially liquefiable. Such CRR (Cyclic Resistance Ratio) correlation curves have almost been developed for clean sands based on field observations and procedures were suggested to account for sands with fines content. Past experience has shown that mineral contents and plasticity of fines are important contributing factors to monotonic and cyclic behavior of sands that contain fines [2]. Up to now, no clear consensus has been received on influence of non-plastic fines on cyclic resistance of sands. Some researchers concluded that increase of fines increases the liquefaction resistance [3-4], while others indicated that the fines decrease the liquefaction resistance [5]. Also in some studies, it has been found that the sand's resistance to liquefaction will initially decreases as the silt content increases and then increases as the silt content continues to increase [6-7].

Although penetration-based methods (i.e., SPT and CPT) are well developed [8], penetration tests may be impractical or unreliable at some sites. Meanwhile, shear wave velocity \( V_s \) offers engineers as a supplementary tool to evaluate liquefaction resistance of soils. Both \( V_s \) and liquefaction resistance are similarly but not proportionally influenced by many of the same factors (e.g., void ratio, state stress, stress history, and geologic age). The advantages of a \( V_s \)-based method have been discussed by...
many researchers [8]. Over the past years, the $V_s$-based procedure for liquefaction assessment has attracted numerous studies and progressed significantly with improved correlations and more complete data bases. The most prevailing approach nowadays is in-situ $V_s$ measurements at sites shaken by earthquakes [9], which follows the framework of the Seed and Idriss [10] simplified procedure and correlates the overburden stress-corrected shear wave velocity ($V_{sI}$) to the magnitude-scaled cyclic stress ratio (CSR) induced by earthquakes. Most of the measured soil parameters for in-situ $V_s$ testing are post earthquake properties and do not exactly reflect the initial soil states before earthquakes. Thus despite their great practical importance, the field CRR-$V_{sI}$ correlations do not furnish insight into the fundamental behavior of liquefiable soils. As it is pointed out by Seed and Idriss [10], with field seismic conditions being properly simulated, the controlled laboratory studies may be used to broaden the applicability of liquefaction criteria, especially for the conditions where little or no field performance data is available. Thereafter many studies have been focusing on this subject on clean sands and sand-silt mixtures [11-20] which demonstrate the validity of laboratory $V_s$-based methods. The CRR$_{field}$-$V_{sI}$ correlations developed in the laboratory and have been compared with the field-based correlations [1]. For example, Rouch et al [14].

This paper describes the results of an experimental study on effect of non-plastic fines on the cyclic resistance obtained based on shear wave velocity and post liquefaction (i.e. pore pressure dissipation and volume change) behavior of sands. The data obtained from this study along with other existing data were transferred to the field and compared to the field performance curves proposed by Andrus and Stokoe [8]. Cyclic triaxial and resonant column tests were conducted on reconstituted samples prepared in laboratory utilizing undercompaction method. The systems used for conducting tests were automated triaxial testing and fixed-free type, torsional resonant column apparatus. A detailed description of the used soils, sample preparation technique and tests procedure are given below.

2. Experimental Program

The study described herein investigates the effects of non-plastic silt contents on the cyclic resistance based on shear wave velocity and post liquefaction behavior of sands using isotropically-consolidated, undrained cyclic triaxial and resonant column tests. Tests were performed on reconstituted samples prepared in laboratory utilizing undercompaction method. The systems used for conducting tests were automated triaxial testing and fixed-free type, torsional resonant column apparatus. A detailed description of the used soils, sample preparation technique and tests procedure are given below.

2.1. Materials Tested

In this study, the Firoozkooh sand (#.161); a uniformly graded sand (SP) with a mean grain size, $D_{50}$ of 0.25mm, coefficient of uniformity, $C_u$ of 1.75 and a specific gravity of 2.67 was used. The soil grains are sub-angular to sub-round in shape. The non-plastic silt used in tests was derived from the fine-grained portion of the Firoozkooh silty sand. Figure (1) shows the grain size distributions of the soils used in this study. Clean sand with three mixtures of sand-silt was used. The mixtures were obtained by mixing respectively 15, 30, and 60% of silt with sand. The specimens were prepared to achieve after-consolidation relative densities of 15, 30, 60 and 75% depending on their silt content. The global void ratios ($e$) and the intergrain void
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ratios ($e_s$ = Sand skeleton void ratio is one that exists in a silty sand if all of the silt particle were removed, leaving only the sand grains and voids to form the skeleton) for the mixtures are presented in Table (1). The vibratory table method (ASTM D4253) was used to determine the minimum void ratio, $e_{\text{min}}$, meanwhile ASTM D4254 procedure was employed to find the maximum void ratio, $e_{\text{max}}$, see Table (1).

2.2. Sample Preparation

In order to obtain a uniform density throughout the specimens, the undercompaction technique [21] was used. The undercompaction method consists of placing each layer at a density slightly greater than the density of the layer below it in order to account for the decrease in volume and increase in density that occurs in the lower layers when the new layer is placed. In this study, the specimens were made in six layers with an undercompaction value of 5%, so that relative density was varied by 1% per layer. To ensure the uniformity of density throughout the specimens’ height, the void ratio distribution within the specimens were obtained. The specimens were solidified and a gelation solution was used [22]. The solidified specimen was then sliced and distribution of void ratio within the test specimen was determined. The measurements revealed that the relative error in achieving the required density throughout the specimens was successfully less than 5% for each layer. For example, the void ratio values of the six layers of sand sample, prepared in 30% relative density was 0.77, 0.76, 0.8, 0.81, 0.8 and 0.81 respectively. In addition, the specimens were prepared in a Plexiglas mold to have better control over the layer’s thickness, see Figure (2). During sample preparation, it was found that forming low density specimens for high silt content (i.e. 60%) materials was impossible, because of excessive collapse during saturation. Thus, high silt content specimens were prepared at high relative densities of 60 and 75%, meanwhile other specimens were prepared at densities of 15, 30 and 60%.

2.3. Test Procedure

The CRR values were measured using an automated stress-controlled cyclic triaxial apparatus. The specimens were tested with a typical diameter of 70mm and a height of 150mm. Small strain shear wave velocity, $V_s$ was also measured using a fixed-free type, torsional resonant column apparatus. The tested specimens were typically 70mm in diameter and 100mm in height. The specimens were saturated with a Skempton B-value in excess of 98%. To facilitate saturation process, carbon dioxide ($CO_2$) was first percolated through the specimens. Then de-aired water was flushed into the specimens. Finally a back pressure of 100kPa was incrementally applied to accelerate saturation rate. Then specimens were isotropically consolidated under an effective confining stress of 100kPa. All relative densities reported herein are based on the after-consolidation void ratios. In cyclic triaxial tests, the specimens were loaded sinusoidally at a frequency of 0.1Hz ASTM D5311 varying deviator stress at

<table>
<thead>
<tr>
<th>Type of Materials</th>
<th>$e_{\text{min}}$</th>
<th>$e_{\text{max}}$</th>
<th>$Dr=15%$</th>
<th>$Dr=30%$</th>
<th>$Dr=60%$</th>
<th>$Dr=75%$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$e_s$</td>
<td>$e$</td>
<td>$e$</td>
<td>$e$</td>
<td>$e$</td>
<td>$e$</td>
</tr>
<tr>
<td>Sand</td>
<td>0.58</td>
<td>0.87</td>
<td>0.83</td>
<td>0.83</td>
<td>0.78</td>
<td>0.78</td>
</tr>
<tr>
<td>Sand+15% Silt</td>
<td>0.41</td>
<td>0.83</td>
<td>0.76</td>
<td>1.08</td>
<td>0.7</td>
<td>1</td>
</tr>
<tr>
<td>Sand+30% Silt</td>
<td>0.32</td>
<td>0.854</td>
<td>0.77</td>
<td>1.52</td>
<td>0.61</td>
<td>1.41</td>
</tr>
<tr>
<td>Sand+60% Silt</td>
<td>0.36</td>
<td>1.259</td>
<td>1.124</td>
<td>-</td>
<td>0.99</td>
<td>-</td>
</tr>
</tbody>
</table>
the appropriate cyclic stress ratio until they were liquefied. In this study, liquefaction was defined and evaluated at initial liquefaction; when the pore pressure in the specimen first equaled the initial confining stress or the specimen reached 5% double amplitude axial strain, whichever occurred first. To observe post liquefaction behavior of the mixtures, once the specimens are liquefied, the cyclic loading phase was terminated and volume change and pore pressure dissipation were measured immediately. Bottom end of each specimen was connected to a volume measuring burette. The top was connected to a pore pressure transducer with no drainage allowed. This setup simulated a one-way drainage condition.

3. Evaluation of Cyclic Resistance and Shear Wave Velocity

Figures (3a), to (3c) presents cyclic stress ratio (CSR) expressed versus number of cycles to liquefaction (N). Cyclic resistance is defined as the cyclic stress ratio causing initial liquefaction in 15 cycles of loading [23]. CRR values and small strain shear wave velocity ($V_s$), shear modulus ($G_{max}$) and damping ratio ($D$) of the sand-silt mixtures are presented in Table (2).

As shown in Figures (3a) to (3c) and Table (2), in very loose and medium dense samples (i.e. $Dr = 15$ and 30%), the cyclic resistance of Firoozkooh sand first, slightly increases with fines content up to 15%, followed by a decrease beyond this value. Similar trend was found by Koester [24] through testing on the mixtures of fine sands and non-plastic silts. In dense samples (i.e. $Dr = 60$%), the cyclic resistance continuously decreases with increase in silt contents. The cyclic resistance of the specimens containing 60% silt with $Dr = 75$% are also shown in Figure (3c). As it was expected, cyclic resistance increased with increase in relative density.

The effect of fines on the shear wave velocity have been less studied and understood. Resonant column tests conducted by Iwasaki and Tatsuoka [25] and Huang et al [15] showed that the small strain-shear modulus, $G_{max}$ and therefore $V_s$ decreased with increase in non-plastic fine content. Table (2) shows that, $V_s$ and Maximum shear modulus ($G_{max}$) in clean sand and sand-silt mixtures increase with increase of relative density. On the other hand, $V_s$ and $G_{max}$ decrease with increase of silt content. On the other hand, in constant relative density, with increasing of fines content up to 30%, $G_{max}$ decreases and then increases with fines content more than 30%.
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Also damping ratio ($D$) decreases with increasing relative densities. In constant relative density, damping ratio decreases with fines content up to 30%, and then it increases a little with fines content up to 60%. Figures (4a) to (4d) present the variations of $CRR$, $Vs$, $G_{max}$ and $D$ versus sand skeleton void ratio, which can be seen that:

Cyclic resistance ratio increases with sand skeleton void ratio for mixtures having up to 15% silt and decreases with further increase of the silt content. In dense specimens (i.e. $Dr=60\%$) $CRR$ continually decreases with sand skeleton void ratio. Shear wave velocity, maximum shear modulus and damping ratio continuously decrease with sand skeleton void ratio and silt content.

As it is shown in Figure (5), with increasing relative densities from 15% to 75%, $Vs$ value increases. Also, in constant relative density, with increasing of the fines content, shear wave velocity decreases.

### 4. Conversion of Laboratory Data to Field

It is of great importance to point out, however, that both of the liquefaction resistance ($CRR$) and shear wave velocity ($Vs$) were obtained in undrained cyclic triaxial and resonant column tests under isotropic consolidation conditions, which are usually different from the in-situ conditions required to be evaluated for design purposes. Therefore, some considerations should be included in applying the laboratory test-based $CRR-Vs$ correlations to in-situ conditions. It is common to correct $CRR$ to in-situ $CRR$ (i.e. $CRR_{field}$) in an approximate manner as follows [26]:

<table>
<thead>
<tr>
<th>Type of Material</th>
<th>$Dr$</th>
<th>$CRR_{Triaxial}$</th>
<th>$Vs$ (m/s)</th>
<th>$G_{max}$ (kG/cm$^2$)</th>
<th>$D$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand</td>
<td>0.15</td>
<td>0.096</td>
<td>182</td>
<td>525</td>
<td>4.7</td>
</tr>
<tr>
<td></td>
<td>0.30</td>
<td>0.132</td>
<td>193</td>
<td>742</td>
<td>4.5</td>
</tr>
<tr>
<td></td>
<td>0.60</td>
<td>0.25</td>
<td>201</td>
<td>855</td>
<td>4.3</td>
</tr>
<tr>
<td>Sand+15%Silt</td>
<td>0.15</td>
<td>0.112</td>
<td>169</td>
<td>452</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>0.30</td>
<td>0.142</td>
<td>181</td>
<td>575</td>
<td>3.8</td>
</tr>
<tr>
<td></td>
<td>0.60</td>
<td>0.23</td>
<td>202</td>
<td>744</td>
<td>3.5</td>
</tr>
<tr>
<td>Sand+30%Silt</td>
<td>0.15</td>
<td>0.061</td>
<td>157</td>
<td>325</td>
<td>3.8</td>
</tr>
<tr>
<td></td>
<td>0.30</td>
<td>0.096</td>
<td>168</td>
<td>446</td>
<td>3.6</td>
</tr>
<tr>
<td></td>
<td>0.60</td>
<td>0.23</td>
<td>189</td>
<td>691</td>
<td>3.4</td>
</tr>
<tr>
<td>Sand+60%Silt</td>
<td>0.60</td>
<td>0.033</td>
<td>164</td>
<td>459</td>
<td>3.6</td>
</tr>
<tr>
<td></td>
<td>0.75</td>
<td>0.093</td>
<td>175</td>
<td>573</td>
<td>3.7</td>
</tr>
</tbody>
</table>
triaxial field 

\[ CR_{RR} \]

\[ \beta = \alpha \]

where \( \alpha \) and \( \beta \) are correction factors. Some equations presented for \( \alpha \) are as follows:

\[ \alpha = K_0 \]

\[ \alpha = 1 + 2K_0 \]

\[ \alpha = 1 + K_0 \]

\[ \alpha = \frac{2(1 + 2K_0)}{3\sqrt{3}} \]

In which \( K_0 \) is the effective earth pressure ratio at rest. Eqs. (2) and (3) were proposed by Seed and Peacock [26] and Eqs. (4) and (5) by Finn et al [27] and Castro [28] respectively. \( K_0 \) was also taken equal to \( (1 - \sin \phi') \), where \( \phi' \) is angle of shearing resistance. For each mixture at desired relative density, \( \phi' \) was determined using monotonic undrained triaxial tests conducted under initial confining stresses of 100, 200 and 300 kPa, see Table (3). Finally, by averaging over the \( \alpha \) values from Eqs. (2) to (5), the desired value of constant \( \alpha \) was determined, i.e. \( \alpha_{\text{mean}} \) in Table (3).

\[ \beta \] is function of relative density [29] and is defined as:

\[ Dr \leq 45\% \Rightarrow \beta = 1.15 \]

\[ Dr > 45\% \Rightarrow \beta = 0.01Dr + 0.7 \]  

Table (3) presents the value of \( \beta \) along with \( CRR_{\text{field}} \) for different mixtures.

On the other hand, the measured \( V_s \) require adjustment for different stress states. As \( V_s \) was widely observed to depend equally on principal stresses in the direction of wave propagation and particle motion [30], \( V_s \) can be expressed as:

\[ V_{sf} = \left( \frac{1 + 2K_0}{3} \right)^{0.25} \]  

Where \( V_{sf} \) is the equivalent field value of laboratory measured \( V_s \). According to common practice [4, 5] the \( V_{sf} \) in Eq. (7) should be further corrected in terms of the in-situ effective overburden stress (\( \sigma'_m \)) as follows:

\[ V_{sl} = V_{sf} \left( \frac{P_a}{\sigma'_m} \right)^{0.25} = V_s \left( \frac{1 + 2K_0}{3} \right)^{0.25} \left( \frac{P_a}{\sigma'_m} \right)^{0.25} \]

where \( V_{sl} \) = overburden stress-corrected velocity; \( P_a = \) atmosphere pressure; and \( \sigma'_m = \) mean effective stress in the laboratory. Table (3) presents the value of \( V_{sl} \) for each mixture.

In Table (3), it is seen that values of \( CRR_{\text{field}} \) increases as the silt content increases up to 15%. With further increase in silt content up to 30%, cyclic resistance ratio decreases in lower relative

### Table 3. \( CRR_{\text{Field}} \) and \( V_s \) for different sand-silt mixtures.

<table>
<thead>
<tr>
<th>Type of Material</th>
<th>( Dr )</th>
<th>( \phi'(%) )</th>
<th>( CRR_{\text{triaxial}} )</th>
<th>( V_s ) (m/s)</th>
<th>( \alpha_{\text{mean}} )</th>
<th>( \beta )</th>
<th>( CRR_{\text{field}} )</th>
<th>( V_{sl} ) (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clean Sand</td>
<td>0.15</td>
<td>0.096</td>
<td>182</td>
<td>0.788</td>
<td>1.15</td>
<td>0.087</td>
<td>170.6</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.30</td>
<td>0.132</td>
<td>193</td>
<td>0.68</td>
<td>1.15</td>
<td>0.103</td>
<td>174.8</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.60</td>
<td>0.25</td>
<td>201</td>
<td>0.63</td>
<td>1.3</td>
<td>0.201</td>
<td>178</td>
<td></td>
</tr>
<tr>
<td>Sand+15%Silt</td>
<td>0.15</td>
<td>0.112</td>
<td>169</td>
<td>0.83</td>
<td>1.15</td>
<td>0.106</td>
<td>160</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.30</td>
<td>0.142</td>
<td>181</td>
<td>0.752</td>
<td>1.15</td>
<td>0.123</td>
<td>167</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.60</td>
<td>0.23</td>
<td>202</td>
<td>0.65</td>
<td>1.3</td>
<td>0.194</td>
<td>181</td>
<td></td>
</tr>
<tr>
<td>Sand+30%Silt</td>
<td>0.15</td>
<td>0.061</td>
<td>157</td>
<td>0.832</td>
<td>1.15</td>
<td>0.058</td>
<td>149</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.30</td>
<td>0.096</td>
<td>168</td>
<td>0.796</td>
<td>1.15</td>
<td>0.087</td>
<td>157</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.60</td>
<td>0.23</td>
<td>189</td>
<td>0.68</td>
<td>1.3</td>
<td>0.203</td>
<td>171</td>
<td></td>
</tr>
<tr>
<td>Sand+60%Silt</td>
<td>0.60</td>
<td>0.033</td>
<td>164</td>
<td>0.69</td>
<td>1.3</td>
<td>0.03</td>
<td>149</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.75</td>
<td>0.093</td>
<td>175</td>
<td>0.66</td>
<td>1.45</td>
<td>0.088</td>
<td>157</td>
<td></td>
</tr>
</tbody>
</table>
As the silt content increases up to 60%, the CRR Field decreases in dense relative density ($Dr = 60\%$) and with the increase of relative density from 60% up to 75%, the CRR Field is increased.

Figure (6) presents the values of $CRR_{Field}$ for different mixtures versus relative densities. It is seen that values of $CRR_{Field}$ increases as the silt content increases up to 15%, with further increase in silt content up to 30%, cyclic resistance ratio decreases in lower relative densities ($Dr = 15\%, 30\%$). As the silt content increases up to 60%, the $CRR$ is decreased in dense relative density ($Dr = 60\%$) and with increase of relative density from 60% up to 75%, the $CRR$ is increased.

As can be seen in Figure (7), in lower relative densities (i.e. $Dr = 15, 30\%$), the values of steady state friction angle ($\phi'$) decreases as the silt content increase up to 30%. With increasing silt content to 60%, $\phi'$ values slightly increase.

### Figure 6. Effects of nonplastic fine content on cyclic resistance ratio ($CRR$) vs. relative densities.

### Figure 7. Variations effects of fines content on steady state friction angle.

### Figure 8. Comparison between converted $CRR_{field}$-$V_{s1}$ data based on the laboratory data from sand-silt mixtures ($FC \leq 5\%$) and the existing field-based correlation of Andrus and Stokoe [8].

### Figure 9. Comparison between converted $CRR_{field}$-$V_{s1}$ data based on the laboratory data from sand-silt mixtures ($5\% < FC < 30\%$) and the existing field-based correlation of Andrus and Stokoe [8].

### 5. Comparison of Converted Laboratory Results with Field-Based Correlations

The $CRR_{field}$-$V_{s1}$ correlations developed from the laboratory results of this study and other studies are compared to the field-based correlations of Andrus and Stokoe [17] for different ranges of fines content ($FC$) as:

1) The laboratory-based correlations for clean sands ($FC \leq 5\%$) that are based on the data from this study, Tokimatsu et al [31]; Rouch et al [14]; Huang et al [15]; Liu et al [18], see Figure (8);

2) The laboratory-based correlations for silty sands with $5\% < FC < 30\%$ that are based on the data from this study, Rouch et al [14]; Huang et al [15] Liu et al [18], see Figure (9); and

3) The laboratory-based correlations for sand-silt mixtures ($FC \geq 35\%$) that are based on the data from this study, Huang et al [14] and Baxter et al [20], see Figure (10).
Figure (8) shows the $CRR_{\text{field}} - V_s^1$ correlation for the clean sand used in this study. As can be seen, converted $CRR_{\text{field}} - V_s^1$ data based on the laboratory data from clean sand is located in the right side of the semi-empirical curve proposed in the simplified procedure for fines content of less than 5%. Similarly, the trends in the laboratory data on sands with 15% fines content shown in Figure (10), is found to be consistent with the liquefaction curve developed by Andrus and Stokoe [8] for $FC = 20\%$ from field performance data. As it is shown in Figure (11), the laboratory-based correlations from this study for $FC = 30$ and $60\%$ plot below the field-based curve for $FC > 30\%$. Therefore, although the results are not sufficient in order to judge well, but based on the results of this study, using the field-based correlation would overestimate the liquefaction resistance of these sand-silt mixtures.

Differences may be due to techniques used for sample preparation in the laboratory which significantly affects the measured cyclic resistance and shear wave velocity, the cyclic stress path generated by uniform cycles of axial stress in a triaxial test which only approximately models an earthquake loading on a soil deposit, the uncertainty of the relationships between the laboratory and field conditions accounted for the correction of cyclic triaxial strength ($CRR_{\text{triaxial}}$) to in-situ cyclic resistance ratios ($CRR_{\text{field}}$), and the assessment of cyclic strength and shear wave velocity in separate soil specimens. On the other hand, the doubt in field performance data may originate from the uncertainties in the plasticity of the fines in the in-situ soils, using post earthquake properties that do not exactly reflect the initial soil states before earthquakes and the assumption that $CRR_{\text{field}}$ is equal to the CSR obtained from Seed and Idriss [10] well known equation.

6. Post-Liquefaction Response of Sand-Silt Mixtures

When saturated loose sand deposits are subjected to shaking during an earthquake, pore water pressure is known to build up leading to liquefaction or loss of strength. The pore water pressure then starts to dissipate mainly towards the ground surface, accompanied by some volume change of the sand deposits which is manifested on the ground surface as settlement. The volume change characteristics of sand due to dissipation of pore water pressures induced by undrained cyclic loading has been studied in the laboratory tests by Lee and Albaisa [32], and Tatsuoka et al [33]. As a result of these studies, it has become apparent that the volumetric strain after liquefaction is influenced not only by the density but more importantly by the maximum shear strain which the sand has undergone during the application of cyclic loads. On the basis of this rationale, an attempt was made by Tokimatsu and Seed [34] to deploy a methodology to predict the post liquefaction settlement of ground. An alternative procedure for estimating ground settlement was explored by Ishihara and Yoshimine [35] by maximum shear strain which is a key parameter influencing the post liquefaction volumetric strain.

This procedure is based on results of extensive laboratory tests. Detailed understanding of post-liquefaction behavior of the sand-silt mixtures, including pore pressure dissipation and densification characteristics can give insight to the behavior of the silty sand. It can also be used for evaluation of post-liquefaction settlement and design of drainage systems to mitigate liquefaction. At present, there is only limited data available on this subject and data is primarily limited to clean sands. Thevanayagam and Martin [36] recently studied the post liquefaction of silty sands. To observe post-liquefaction behavior of the mixtures, once specimen liquefied, the cyclic loading phase was terminated and volume change and pore pressure dissipation were measured immediately. Volume change was measured from the bottom end of the specimen, whilst pore pressure dissipation was measured from the top end of the specimen. Figure (11) shows the variation of dissipated post-liquefaction pore pressure versus
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As it can be seen, when silt content is raised, the required time for dissipation of the generated pore pressure is also increased. For example, specimen containing 30% fines require more time to dissipate all the generated pore pressure with respect to specimen without silt content (other specimens have the same behavior). The observed trend is logical, since void ratio and subsequently permeability decrease when silt content is raised.

Figure (12) shows variations of volumetric strain versus relative densities for all of specimens. An interesting feature of post-liquefaction behavior which can be found is that higher final volumetric strain would be attained when silt content is raised. This means that increasing silt content causes a looser fabric to be formed.

Figure (13) shows the post-liquefaction densification for sand-silt mixtures in \( \sigma'_0 = 100kPa \) and relative density of 60% (CSR about 0.21). The figure presents variation of the volumetric strain (i.e. volume of water expelled out of the specimen divided by the end of consolidation volume of specimen) versus time for sand-silt mixtures. Also, the figure shows that volumetric strain increases with time, reaching to a constant value (i.e. final volumetric strain). However, for specimens containing fines content, because of lower permeability, the time needed to reach the final volumetric strain is more (other specimens have the same behavior). Generally, it can be seen that in all the specimens, as the silt content increase up to 30%, the final volumetric strain increases.

Diagrams in Figures (11) to (13) show that increasing silt content causes a looser fabric to be formed and in this regard, sand skeleton void ratio may describe this behavior better.

7. Conclusion

The results of an experimental study on the liquefaction behavior of reconstituted specimens of non-plastic sand-silt mixtures at different fines content and relative densities based on shear wave velocity were presented. Cyclic resistance, shear wave velocity and post-liquefaction behavior of the mixtures were studied. The results of this study along with other studies were compared to field based \( CRR-V_{s1} \) curves prepared by Andrus and Stokoe [8]. The following conclusions regarding the effects of non-plastic fines on the liquefaction susceptibility of sands can be drawn from this study:

- When the soil is loose or medium dense (i.e. \( Dr = 15 \) and \( 30\% \)), the \( CRR \) of Firoozkooh sand
increases slightly with fines content up to 15%, followed by a decrease beyond this value. In dense samples (i.e. $Dr = 60\%$), the cyclic resistance ratio of Firoozkooh sand continuously decreases with increase of silt content. In dense samples (i.e. $Dr = 60\%$), the cyclic resistance ratio of Firoozkooh sand continuously decreases with increase of silt content. 

- Data obtained on the cyclic resistance, shear wave velocity and dynamic properties of the mixtures evidently show that, $CRR$, $V_s$, $G_{max}$ and Damping ratio have good correlation with skeleton void ratios. It can be generally concluded that increase in fines content leads to decrease in cyclic strength, shear wave velocity, maximum shear modulus and damping ratios.

- In conversion of laboratory data to field condition, results show that the $CRR_{field}$-$V_s$ correlation for the clean sand lie closely to the semi-empirical curve proposed in the simplified procedure for fines content less than 5% by Andrus and Stokoe [8]. This trend is also observed in the converted laboratory data of this study on sands with 15% fines content and results are consist with the liquefaction boundary curves developed by Andrus and Stokoe [8] for $FC = 20\%$. The $CRR_{field}$-$V_s$ values for $FC = 30$ and $60\%$ in present research are below the field-based curve for $FC \geq 35\%$, means that field-based correlation overestimates the liquefaction resistance of these sand-silt mixtures in comparison with the present study.

- When silt content is raised to 30% and more, the required time for dissipation of the post-liquefaction pore pressure is increased. Furthermore, increase of fines content to 30% and more leads to more final volumetric strains without any dependence on maximum shear strains.

- In general, when fines content is raised to 30% and more, the stability of the mixture fabric seems to be reduced. Data obtained from cyclic resistance, shear wave velocity, maximum shear modulus and damping ratio of mixtures evidently show that, increasing of fines content to 30% and more lead to less $CRR$, $V_s$, $G_{max}$ and $D$.

Acknowledgements

This research was conducted in the International Institute of Earthquake Engineering and Seismology (IIEES) and supported under the Contract No. 6712, which is gratefully appreciated.

References


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