LIQUEFIABLE SAND BEHAVIOR UNDER DIFFERENT APPLIED CYCLIC STRAIN AMPLITUDE IN CYCLIC TRIAXIAL TEST

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ABSTRACT
The behavior of dynamic properties of saturated loose clean sand tested in strain control cyclic triaxial apparatus with various applied cyclic strain amplitude were investigated. The number of cyclic strain required to cause liquefaction was determined. Besides, Young’s modulus and shear modulus were calculated in addition to shear wave velocity and damping ratio. The test results present a good relation for the effect of the applied cyclic strain amplitude. The results indicate that the effect of increasing the applied cyclic strain amplitude to 𝜀=1.0% significantly decreases the number of cycle to cause liquefaction and soil moduli, while on effect has been occurred after increasing the applied cyclic strain amplitude more than 𝜀=1.0%.

Keywords: Cyclic strain, Cyclic triaxial test, Liquefaction, Soil moduli

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1. INTRODUCTION
Comprehension the dynamic cyclic response of granular soils is a key factor for many geotechnical applications especially structures exposed to earthquake loadings. Structures may suffer damage because of settlement of ground and lateral spreading of sloping ground. The causes of these failures can be attributed to liquefaction of soil. Liquefaction is a phenomenon when saturated cohesionless soil losses strength due to increasing pore water pressures and therefore shear strength is reduced because of dynamic loading (Biswa
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Naik 2010) [1]. Over the previous few decades, comprehensive studies have been conducted to evaluate dynamic properties and liquefaction resistance of soils exposed to earthquake loading in the laboratory. In undrained cyclic loading test, the sand matrix or skeleton can tend to contract under the cyclic loads, the resulting rearrangement of sand particles instead transfers normal stresses from the sand matrix to the pore water pressure (Idriss and Boulanger 2008) [3]. The liquefaction response of saturated fine sand under cyclic stress controlled and strain controlled triaxial test were studied and compared by (Du and Chian 2015) [2], if cyclic stress amplitude is very low, the soil appears to get compacted with number of cycles. Shear modulus gradually increases without significant generation of excess pore pressure, if cyclic stress amplitude is high, shear modulus decreases due to strain softening. In either condition, the accumulated shear strain of the soil increases with number of cycles. For strain controlled testing shear modulus decreases with number of cycles despite maintaining at constant cyclic shear strain amplitude. Cyclic stress ratios (CSR) and number of cycles (N) significantly affect the development of excess pore water pressure and the associated dynamic soil properties, the shear modulus measured from each cycle with accumulated shear strain shows conventional degradation pattern with the increase in shear strain corresponding to (CSR), while the damping ratio showed non-conventional degradation pattern beyond a shear strain (Kumar et al. 2015) [4].

In the present paper, cyclic strain control triaxial test have been carried out to present the behavior of the clean loose sand of relative density (Dr=33%) under consolidation pressure of (σ'c=30 kN/m²), then cycled by applying a set of cyclic strain (ε=0.5, 1.0, 2.0 and 5.0 %), of cyclic frequency of (f=2 Hz).

2. STRAIN CONTROL CYCLIC TRIAXIAL EXPERIMENTAL WORK

The soil used for the specimen tests was saturated loose clean sand. The properties of implemented sand are summarized in Table 1. Sand specimen of 38mm diameter and 76mm in height was prepared for the triaxial tests in a dry state and subsequently saturated with de- aired water via the backpressure inlet. Simultaneously the cell pressure and back pressure were increased to 110 and 100 respectively in steps of 30 kN/m² with specimen drainage valve that connected to burette was kept opened to ensure that all air bubbles were dissolved in water. Then, back pressure valve was closed and after the pore water pressure reading is stabled the cell pressure was increased according to the desired consolidation pressure (σ'c), in this study 30 kN/m² was adopted. Pore water pressure started to increase, when the cell pressure increased and the Skempton parameter B-value was monitored, which is the ratio of change in pore water pressure (∆u) to consolidation pressure. The saturation of specimen considered to be fully when B-value exceeds 0.95. After fully saturation state, the consolidation step was followed by opening the burette valve. Water starts to dissipate from specimen to burette and stabilized in few seconds. Readings of specimen displacement and volume change were recorded. Then, burette valve was closed and back pressure valve opened until pore water pressure stabilized. Finally, desirable cyclic strain has been applied. As well as measurement of strain and pore water pressure has been taken.
Table 1 Physical sand properties

<table>
<thead>
<tr>
<th>Soil Property</th>
<th>Loose Sand</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relative density, $D_r$ (%)</td>
<td>33</td>
</tr>
<tr>
<td>Dry unit weight, $\gamma_d$ (kN/m$^3$)</td>
<td>15.65</td>
</tr>
<tr>
<td>Total unit weight, $\gamma_t$ (kN/m$^3$)</td>
<td>19.55</td>
</tr>
<tr>
<td>Water content, $W_c$ (%)</td>
<td>25</td>
</tr>
<tr>
<td>Specific gravity, $G_s$</td>
<td>2.65</td>
</tr>
<tr>
<td>Maximum. void ratio, $e_{\text{max}}$</td>
<td>0.757</td>
</tr>
<tr>
<td>Minimum. void ratio, $e_{\text{min}}$</td>
<td>0.467</td>
</tr>
<tr>
<td>Void ratio, $e$</td>
<td>0.661</td>
</tr>
<tr>
<td>Fine content, %</td>
<td>4.0</td>
</tr>
<tr>
<td>Effective size, $D_{10}$ (mm)</td>
<td>0.162</td>
</tr>
<tr>
<td>Mean size, $D_{50}$ (mm)</td>
<td>0.394</td>
</tr>
<tr>
<td>Soil classification (USCS)$^1$</td>
<td>Poorly-graded sand, (SP)</td>
</tr>
<tr>
<td>Permeability coefficient$^2$, $k$ (cm/sec)</td>
<td>1.21x10$^2$</td>
</tr>
<tr>
<td>Friction angle, $\varphi$</td>
<td>34°</td>
</tr>
<tr>
<td>Cohesion, $c$ (kN/m$^2$)</td>
<td>0</td>
</tr>
</tbody>
</table>

USCS: Unified Soil Classification System

For sand constant head permeability test

3. EXPERIMENTAL TEST RESULTS

Figure 1 to Figure 4 presents the results for the tests, it is observed that, the plot of the strain along the duration of the test, shows that the strain is cycled between (+$\epsilon$ compression and -$\epsilon$ extension). Applied strain remains constant along the test duration. Plot of cyclic stress ($\sigma_d$) shows that the maximum amplitude of the stress is at the beginning of the test, then decreased to a certain value as the strain cycled, then the cyclic stress amplitude ($\pm \sigma_d$) remains constant along that test to a value found to be equal to the minor principal effective stress ($\sigma_{3\prime}$) and the major principal effective stress reduced entirely ($\sigma_{1\prime}$=0), which means that the sand particles lost all its shear strength. The initial (maximum) value of the cyclic stress amplitude found to be about ($\sigma_d\text{ (initial)}=120, 180, 250$ and $380$ kN/m$^2$) for the specimen tested by cyclic strain of ($\epsilon=0.5, 1.0, 2.0$ and $5.0$ %) respectively, then these values decreased to be equal to the minor principal effective stress ($\sigma_{3\prime}$) after a certain number of cycles. The ($NC_{\sigma}$) is the number of strain cycles required to make ($\sigma_d = \sigma_{3\prime}$), it is found to be ($NC_{\sigma}$=54, 26, 17 and $24$ kN/m$^2$) for the specimen tested under the cyclic strain of ($\epsilon=0.5, 1.0, 2.0$ and $5.0$ %) respectively.

Also it is observed that, the excess pore water pressure ratio ($r_u$) increases during the strain cycling. These increments are tightly related to the decrement of the deviator stress amplitude ($\sigma_d$). Besides, the number of strain cycles required to cause initial liquefaction for cyclic strain of ($\epsilon=0.5, 1.0, 2.0$ and $5.0$ %) found to be ($NC_l=16, 4, 4$ and 3) respectively.

(a) Strain ($\epsilon$) vs. no. of cycles (NC)

(b) Strain ($\epsilon$) vs. no. of cycles (NC)
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Figure 1 Test results for applied strain \( \varepsilon = 0.5\% \)

Figure 2 Test results for strain \( \varepsilon = 1.0\% \)
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(a) Strain (ε) vs. no. of cycles (NC)

(b) Stress (σd) vs. no. of cycles (NC)

(c) Excess pore water pressure ratio vs. no. of cycles

(d) Stress (σd) vs. strain (ε)

(e) Deviator stress (q') vs. mean effective stress (p')

(f) Soil moduli (E and G) vs no. of cycles

(g) Shear wave velocity vs. no. of cycles

(h) Damping ratio vs. no. of cycles

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Moreover, plot of \((q' - p')\) also presents. Since all specimen tested under different applied cyclic strain has reached the initial liquefaction condition, the amplitude of deviator stress \((q')\) value show a linear decrement while the mean effective stress \((p')\) values reduced gradually and reached the zero value. The slope of the \((q' - p')\) plot represents the failure envelope line.

Furthermore, the stress-strain plot shows that the hysteresis loop center \((\varepsilon = 0, \sigma_c = 0)\) has not changed along the test duration. As the strain cycled the hysteresis loop become smaller and smaller, then it remains constant when the deviator stress reduction ended, this lead to decrease the soil moduli \((E\) and \(G\)) and shear wave velocity \((V_s)\) along the test duration, these reductions closely related to the reduction of the deviator stress amplitude \((\sigma_d)\).

The damping ratio \((D)\) increasing as the strain cycles, this increments also closely related to the reduction of the deviator stress amplitude \((\sigma_d)\).

Figure 5a present the relation of the initial value of the cyclic stress amplitude \((\sigma_d (\text{initial}))\) against the applied cyclic strain, it is observed that as the applied cyclic strain logarithmically increase the \((\sigma_d (\text{initial}))\) increases linearly.

Figure 5b presented, \((NC_\sigma)\) against the applied cyclic strain, it can be concluded that, the \((NC_\sigma)\) value decreased sharply as the applied cyclic strain increased, these decrements corresponding to the cyclic strain of \((\varepsilon = 0.5\%)\) are about \((52\%, 69\% \text{ and } 56\%)\) for \((\varepsilon = 1.0, 2.0 \text{ and } 5.0\%)\) respectively.

The initial liquefaction occurs in each applied cyclic strain as shown in Figure 5c

Figure 5d display the variation of the \((NC_i)\) with the applied strain, the figure shows that the relation is similar to the relation of \((NC_\sigma)\). The \((NC_i)\) value decreased sharply as the applied cyclic strain increased, these decrements corresponding to the cyclic strain of \((\varepsilon = 0.5\%)\) are about \((75\%, 75\% \text{ and } 81\%)\) for \((\varepsilon = 1.0, 2.0 \text{ and } 5.0\%)\) respectively. In accordance with the presented results, previous study by (Sitharam et al. 2004) [5] have demonstrated the same finding.

Figure 5e, f and g presents the variation of the soil average moduli \((E\) and \(G\)) and shear wave velocity \((V_s)\) with the applies cyclic strain. It can be concluded that, the soil average moduli \((E\) and \(G\)) value decreased sharply as the applied cyclic strain increased, these decrements corresponding to the cyclic strain of \((\varepsilon = 0.5\%)\) are about \((74\%, 87\% \text{ and } 79\%)\) for \((\varepsilon = 1.0, 2.0 \text{ and } 5.0\%)\) respectively. Whereas, the shear wave velocity \((V_s)\) value also decreased sharply as the applied cyclic strain increased, corresponding to the cyclic strain of \((\varepsilon = 0.5\%)\) these decrements are about \((48\%, 63\% \text{ and } 54\%)\) for \((\varepsilon = 1.0, 2.0 \text{ and } 5.0\%)\) respectively.

Figure 5h presented the variation of the damping ratio with applied cyclic strain. It is shown that, the damping ratio \((D)\) value decreased as the applied cyclic strain increased, these decrements corresponding to the cyclic strain of \((\varepsilon = 0.5\%)\) are about \((19\%, 47\% \text{ and } 65\%)\) for \((\varepsilon = 1.0, 2.0 \text{ and } 5.0\%)\) respectively.
4. CONCLUSION

In this study, strain control cyclic triaxial tests were carried out on saturated loose clean sand to examine the influence of the applied cyclic strain amplitude on the dynamic properties of the tested sand, the test results present a good relation for that influence. It can be concluded that, the number of cycle to cause initial liquefaction decreased sharply as the applied cyclic strain increased to $\varepsilon=1.0\%$ after this value, there is no effect of the applied cyclic strain amplitude on the number of cycle to cause initial liquefaction. The soil average moduli value decreased sharply as the applied cyclic strain increased to $\varepsilon=1.0\%$, then as the applied cyclic strain amplitude increases the soil moduli remain approximately constant. The shear wave velocity and the damping ratio values also decreased as the applied cyclic strain increased.

REFERENCES


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