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Excess Pore Pressure Generation in Sand Under Non-Uniform Strain Amplitudes

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ABSTRACT

Two sets of strain-controlled cyclic triaxial tests were conducted to investigate soil liquefaction of clean sands. The first set involved conventional uniform strain amplitude cyclic tests, while the second set examined non-uniform strain amplitude cyclic tests. Comparison was made between the two sets of results with respect to the generation of excess pore pressure and degradation of shear modulus with number of cycles. In the case of uniform strain controlled cyclic tests, lower confining pressure or larger strain amplitude would produce more rapid generation of excess pore pressure. However, in the case of non-uniform strain controlled tests, lower excess pore pressure was generated at cycles with higher strain amplitude. Such counter-intuitive phenomenon is described in this paper through q-p' paths.

Introduction

Damage caused by soil liquefaction during earthquakes due to loss of shear strength of the soil has been extensively studied with laboratory cyclic tests over the past few decades. The cyclic triaxial test is the most widely used laboratory test to evaluate the liquefaction potential of a soil. When a specimen is subjected to repeated shear loading, the sand particles tend to rearrange their stacking into a denser state. When drainage is prevented (similar to field conditions during an earthquake), this would result in generation of pore pressures and loss of effective stresses. Most of these experiments involve applying cyclic loads of uniform amplitude. In the field, the shear stress induced on a soil element in the ground during an earthquake varies non-uniformly in magnitude and frequency. Due to experimental difficulties, few investigations have been conducted using non-uniform or irregular loading patterns. Seed and Idriss (1971) proposed that the effect of irregular earthquake loading can be modeled in the laboratory by a number of uniform shear stress cycles with a magnitude equal to 65% of the maximum shear stress achieved during the field loading sequence. This type of equivalent uniform stress cycle concept has been adopted extensively in practice but it lacks analytical or experimental verification. lshihara and Yasuda (1972) first performed irregular triaxial tests on saturated sand to simulate

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the more representative loading induced during earthquakes. Two loading patterns were classified from their study: shock type loading (maximum stress builds up in a few cycles) and vibration type loading (maximum stress builds up gradually). Their tests showed that the soil liquefied more easily under shock type loading with the same maximum stress.

In order to supplement the sparse experimental data on non-uniform cyclic loading, a comparison between conventional uniform strain controlled cyclic triaxial tests and non-uniform cyclic triaxial tests were carried out. This paper presents the findings from both uniform and non-uniform strain amplitude testing of sand using a cyclic triaxial apparatus. Specimens of similar relative density (RD) at about 38% were prepared. Confining pressures were set at either 40kPa or 80kPa. In the case of uniform cyclic triaxial tests, cyclic axial strain amplitudes of 0.8mm, 1.0mm and 1.2mm were adopted. As for non-uniform tests, two different shear strain amplitudes were applied in each tests, namely 0.35%-0.43%, 0.35%-0.52% and 0.35%-0.66%. The results of this study will aim to contrast their differences in development of pore pressure, stress and strain under dynamic loading conditions. Specific to the tests carried out in this study, the term 'non-uniform amplitude' would refer to the loading pattern as defined graphically in Figure 1. It refers to tests with two different axial strain amplitudes that alternate every 5 cycles.

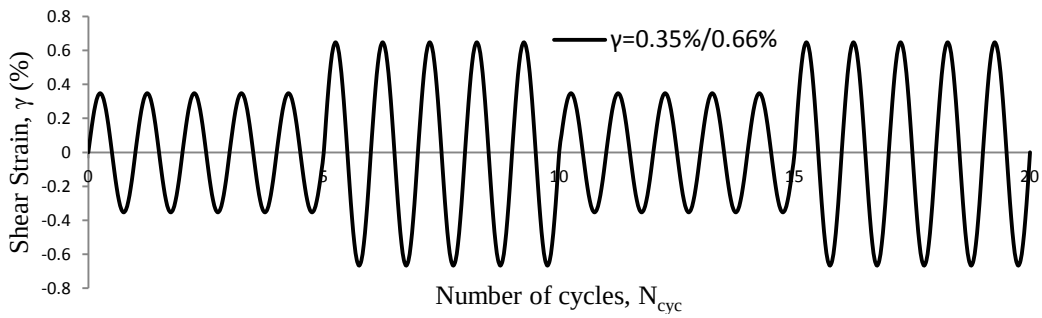


Figure 1. Illustration of non-uniform strain-controlled triaxial cyclic test

Experimental Setup

Soil Properties

W9 sand, fine silica sand supplied by Riversands Pty Ltd, Brisbane was used. Physical properties of the sand are listed as follow: Φ_{crit} =30°, D_{10}=0.22mm, D_{50}=0.26mm, D_{60}=0.3mm, G_s=2.63, e_{max}=1.02 and e_{min}=0.529.

Experimental Setup

Undrained, strain-controlled, cyclic triaxial tests were performed on W9 sand at shear strains varying from 0.35% to 0.66%, during which range the experiment is high sensitive to liquefaction. A cylindrical soil specimen of 38mm diameter and 76mm in height were prepared in a watertight rubber membrane inside a triaxial chamber, where it is later subjected to a confining pressure during testing. The soil samples were initially prepared in a dry state and subsequently saturated with de-aired water via the backpressure inlet. An axial load is applied on the top of the specimen by a load rod. Specimens are consolidated isotropically (equal axial and

radial stress). Tubing connections to the top and bottom specimen platens permit flow of water into and out of the soil during saturation and consolidation. Skempton's pore pressure parameter B ($\Delta u/\Delta\sigma_3$) was checked to ensure that the sample was fully saturated. A value of B=0.95 or greater would indicate that saturation is complete. Thereafter, the specimen is subjected to a sinusoidal cyclic deviatoric loading by means of the load rod connected to the specimen top platen. The deviator stress (q) is generated by applying an axial strainε_a to the soil. Shear strain is evaluated from the applied axial strain based on the following equation:

$$\gamma_c = \frac{2}{3}\,(\in_a + \in_r) \tag{1}$$

where $\in_a$is axial strain and $\in_r$ is radial strain. The cyclic load, strain and pore water pressure development with time were monitored. The test is conducted under undrained condition to represent the near undrained condition in the field during earthquake or other rapid dynamic loading. The cyclic loading generally results in an increase in pore water pressure in the specimen, resulting in a proportional decrease in the effective stress of the soil and accordingly an increase in excess pore pressure ratio, defined as ratio between excess pore pressure and effective confining pressure, r_u. Failure is defined as the point when the excess pore pressure ratio is near 1.0 (shown in Figs.2,3,6 as dashed line). Details of the cyclic triaxial tests are listed in Table 1.

Table 1. Details of cyclic triaxial tests

Test No	**Test Parameters**			
	Relative Density (%)	Cyclic Axial Amplitude (mm)	Cyclic Shear Strain (%)	Confining Pressure (kPa)
G1	38.3	0.8	0.35	40
G2	37.7	0.8	0.35	80
G3	38.6	1.0	0.43	40
G4	37.9	1.0	0.43	80
G5	38.3	1.2	0.52	40
G6	38.5	1.2	0.52	80
G7	37.8	0.8/1.0	0.35/0.43	40
G8	38.2	0.8/1.2	0.35/0.52	40
G9	38.6	0.8/1.5	0.35/0.66	40

Experimental Results

Uniform Cyclic Test Results

The test results contain first 50 cycles of cyclic strain-controlled tests. All samples attained near

full liquefaction at the end of 200 cycles (excess pore pressure ratio above 0.9). In the uniform cyclic strain tests, it was observed that larger applied strain amplitude produced higher rate of generation of excess pore pressure as shown in Figure 2. The specimen will take less number of cycles to reach full liquefaction, similar to the general observations of previous researchers such as Dobry (1985). From Figure 3, it is apparent that the confining pressure has similar effect on pore pressure generation i.e., as the applied confining pressure decreases, the number of cycles to full liquefaction increases.

Figure 4 shows the q-p' stress path of a typical undrained cyclic strain-controlled test from this study, overlaid with the "characteristic threshold" line (CL) and "Failure" line (FL) introduced by Luong and Sidaner (1981). When the stress path lies within the CL boundary, no irreversible volume change is seen; on the other hand, stress performed above CL region result in dilative response. During the process of liquefaction, maximum effective mean stress decreased with increasing number of cycles, corresponding to generation of excess pore pressure as shown in Figure 2 and 3. When the stress path surpassed the CL, soil dilate and generate a suction force leading to an increase in effective mean stress and hence a decrease in r_u as shown in Figure 2, which was aligned with the increase in p' in Figure 5. For the next half cycle, the specimen experienced tension and an opposite result was observed. Double frequency of excess pore pressure and the "butterfly pattern" trend can be observed as shown in Figures 2 and 4(b), respectively. These details were discussed by Chian (2012).

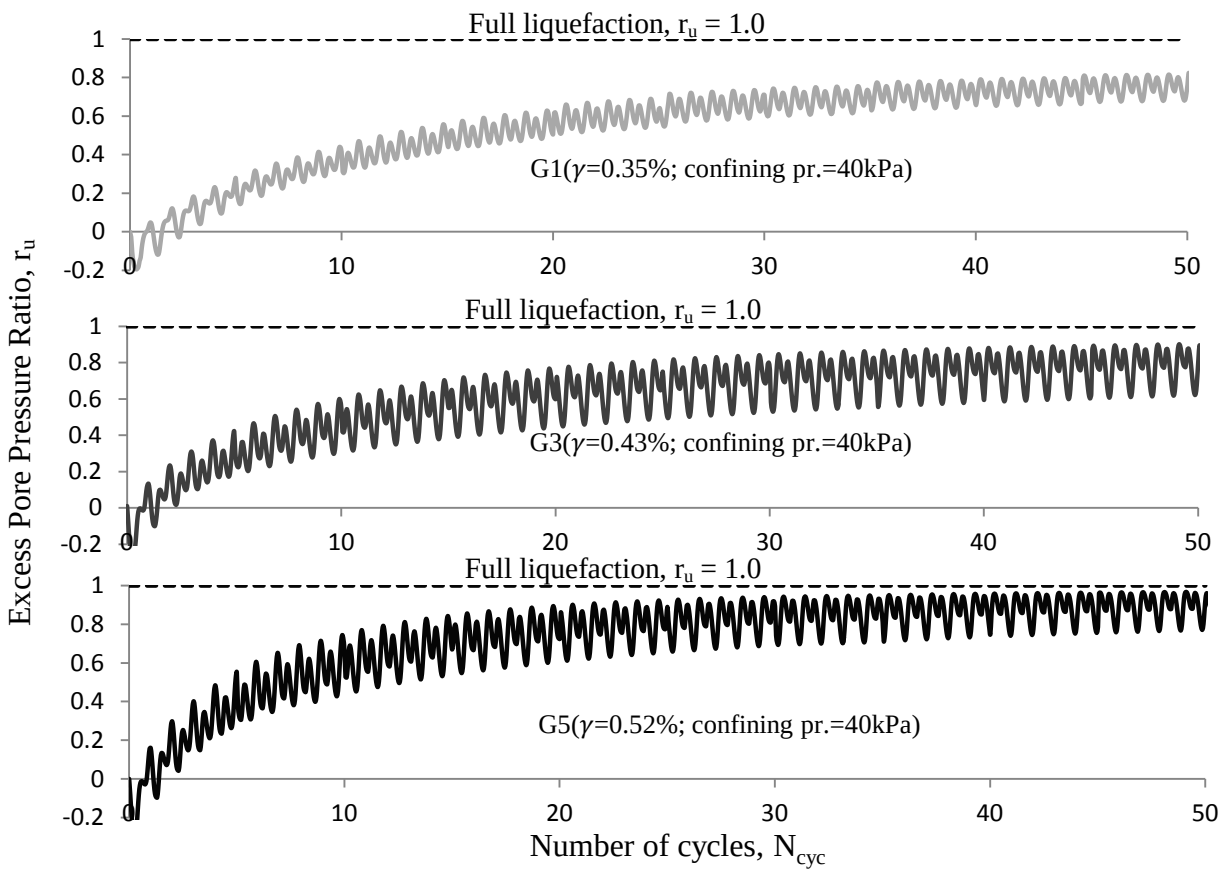


Figure 2. Relationships between excess pore pressure ratio and cycle ratio of W9 sand for RD 37%-39% at various strains (effect of γ.)

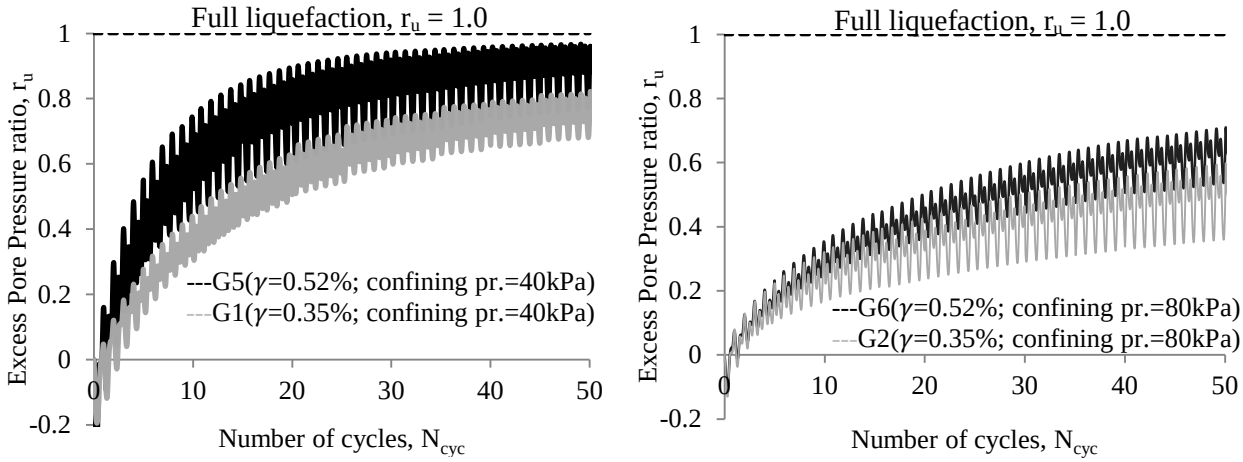
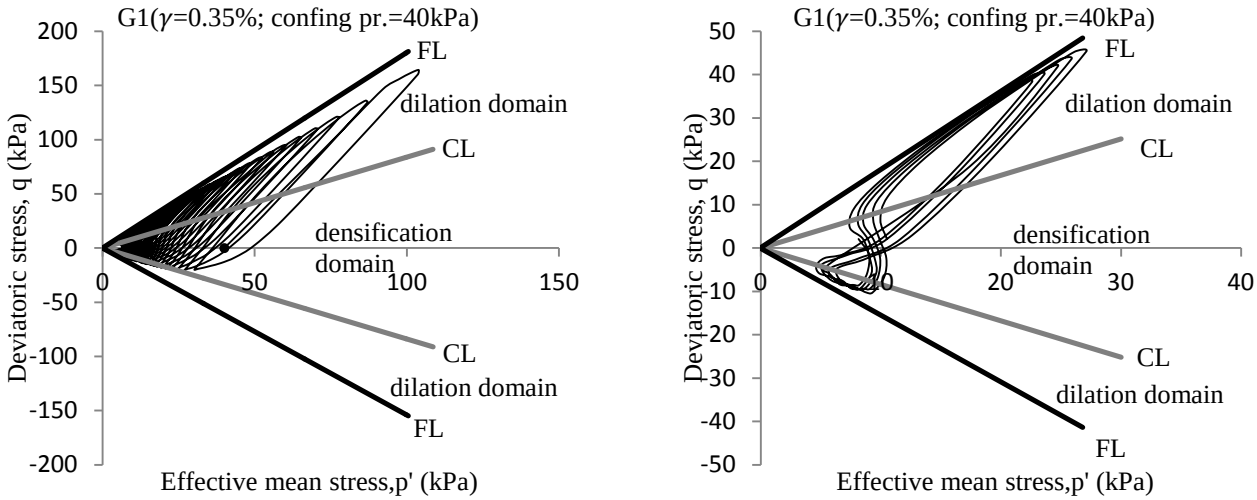


Figure 3. Effect of confining pressure on the pore pressure generation with number of loading cycles at shear strain of 0.35% and 0.52%

Results on the effect of shear strain and confining pressure on shear modulus are shown in Figure 5. A larger cyclic shear strain amplitude produced a lower apparent shear modulus in the first cycle of loading (Figure 5(a)). This is similar to the trend of typical shear modulus degradation curves with increasing shear strain. Another observation can be made with the same figure as well. Shear modulus decreases with increasing loading cycles within the each test (Figure 5(b)), due to the generation of excess pore pressure. In addition, initial confining pressure has an effect on the shear modulus degradation as higher confining pressure would yield higher initial shear modulus. However when the soil approaches full liquefaction with increasing number of cycles, the shear modulus decreased significantly to values similar to each other, hence eliminating the effect of confining pressure to some extent.



(a) 1-50cycle (b) 46-50 cycle

Figure 4. Typical q-p′ space plot of cyclic uniform strain controlled test, Test G1

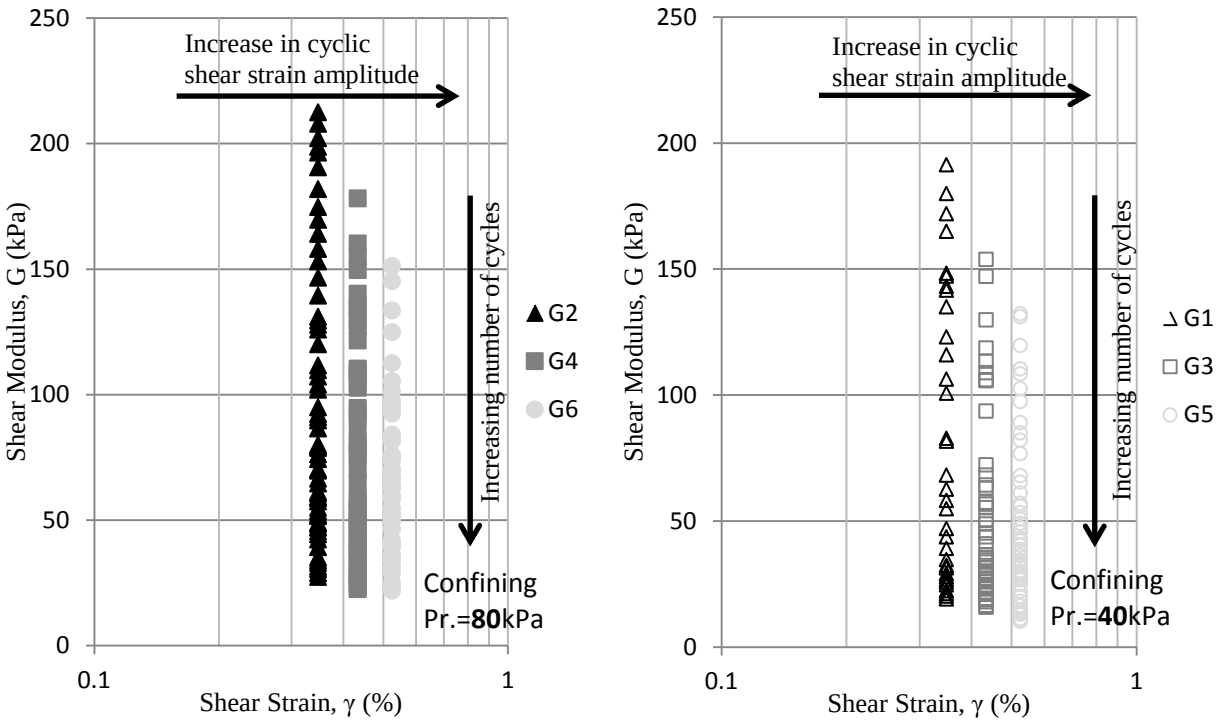


Figure 5. Shear Modulus, G (kPa) vs Shear Strain, γ (%) under two different Confining Pressure in uniform strain-controlled test

Non-Uniform Cyclic Test Results

The effect of the non-uniform amplitude on the accumulation rate of excess pore pressure was studied in a series of tests with the lower shear strain (*γ)* kept constant at 0.35% and other strain amplitudes varying from 0.43% to 0.66%. It is evident in Figure 6 that with larger shear strain amplitude of the latter, the lower the number of cycles required to attain excess pore pressure ratio exceeding 0.9. At the end of first 5 cycles in non-uniform tests, excess pore pressure build up were similar due to identical shear strain as in the uniform test. In the following five cycles, the tests with larger shear strain pair produced higher rate of excess pore pressure generation (i.e. maximum excess pore pressure of 0.35%-0.66% pair is greater than the 0.35%-0.43% pair for cycles 5 to 10). However, interestingly, despite being capable of generating higher excess pore pressure for larger cyclic strain amplitude tests as shown in Figure 3, the larger strain amplitude sections in non-uniform strain tests result in lower excess pore pressures as compared to the lower strain amplitude sections as shown in Figure 6. This is owing to the larger dilation as inferred by the larger "butterfly loops" denoted in light grey in Figure 7. The larger "butterfly loops" caused by the larger strain amplitude cycles transit further upwards along the failure plane (FL) shown in Figure 4 than the lower cyclic strain amplitude until the loading was reverse. As a result, a higher effective mean stress was produced and hence a lower excess pore pressure in the soil. The lower maximum excess pore pressure for the larger strain amplitude cycles in Figure 6 is also the result of the higher effective mean stress at low deviatoric stress in Figure 7, therefore confirming the counter-intuitive phenomenon of lower excess pore pressures at larger strain

amplitudes. This could have implications to the use of equivalent uniform stress cycle concept to represent the non-uniform loading nature of earthquakes in the field.

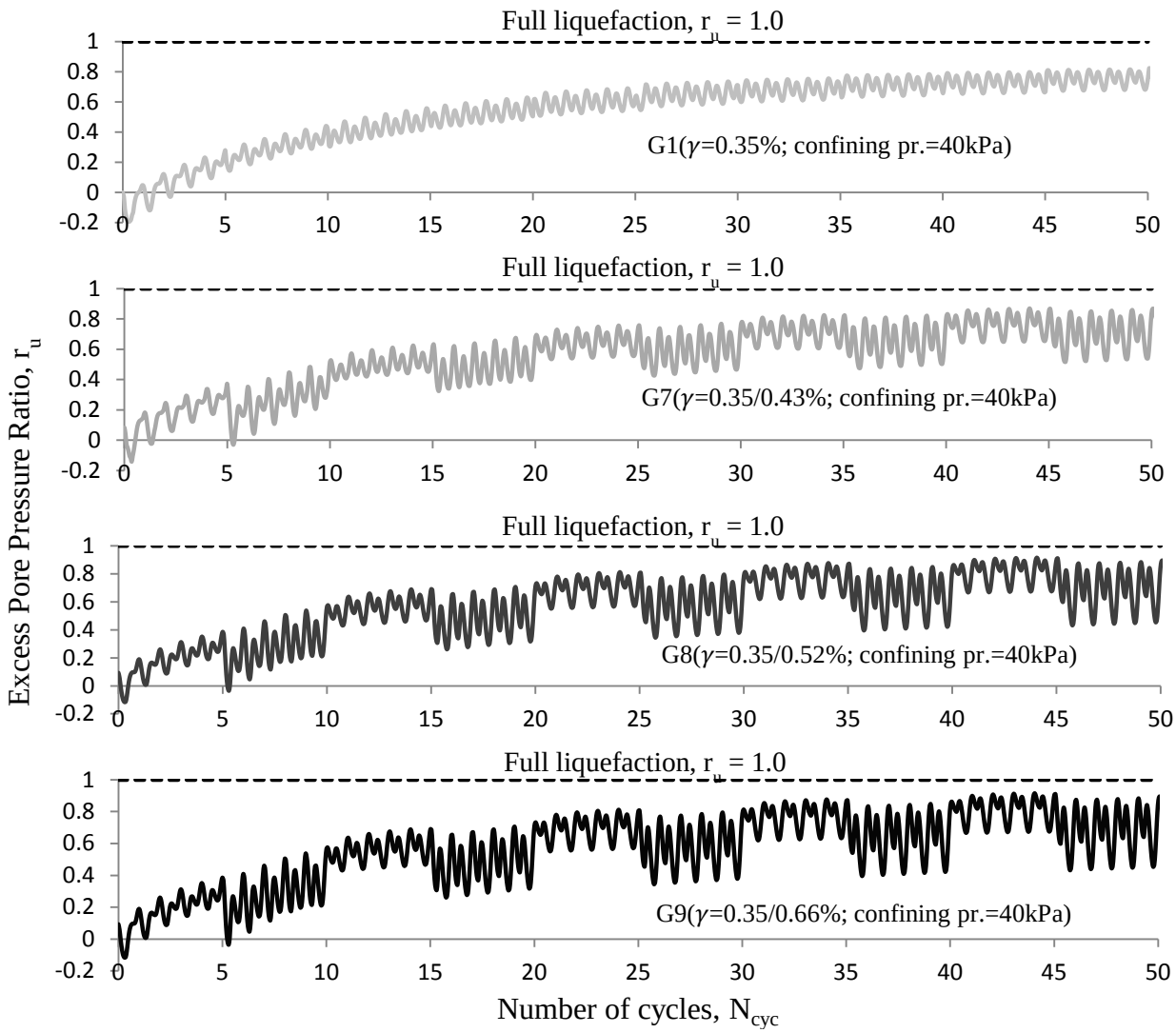
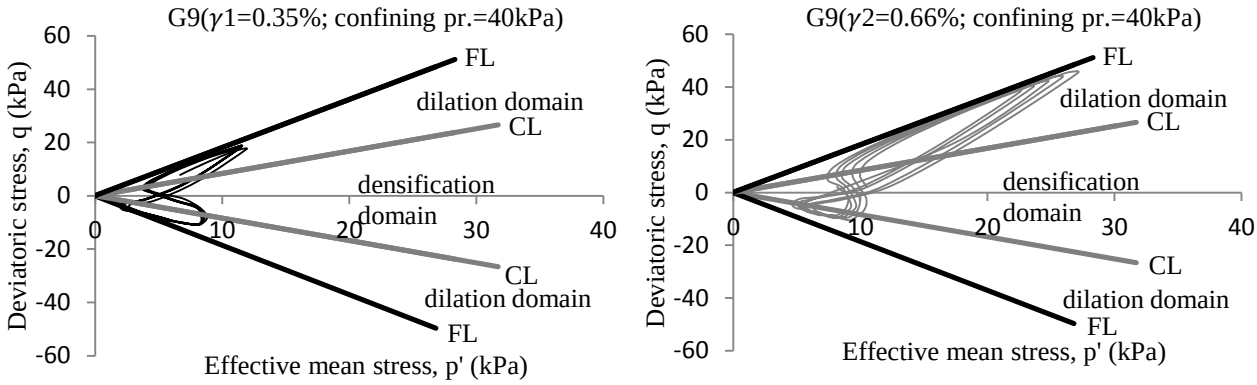


Figure 6. Excess Pore Pressure Ratio behavior of sand under non-uniform cyclic test

Conclusions

Undrained cyclic strain-controlled triaxial tests on clean W9 sand subjected to uniform and non-uniform strain amplitudes were conducted. For uniform strain amplitude tests, lower confining pressure or higher shear strain amplitude produce higher rate of excess pore pressure generation. Shear modulus degradation curves at different number of loading cycles were obtained as the sand sample approaches full liquefaction. In the case of non-uniform strain amplitude tests, a counter-intuitive phenomenon of lower excess pore pressures at larger strain amplitudes were observed. Further analysis with the q-p' space showed that the phenomenon was due to the larger dilative response at higher strain amplitude. There is therefore a need to further assess the impact of adopting equivalent uniform stress cycle concept to represent the non-uniform cyclic loadings observed in real earthquake events.



(a)41-45cycles (b) 46-50 cycles

Figure 7. Typical q-p′ space plot of cyclic non-uniform strain controlled test, Test G9

Acknowledgments

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