Multiple Cyclic Loading Response of Loose Air-Pluviated Fraser River Sand

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SUMMARY:
Multiple cyclic loading response of air-pluviated Fraser River sand was investigated using constant-volume direct simple shear tests to provide input for numerical models that simulate earthquake shaking in centrifuge model tests. The tests were carried out on specimens with a relative density of about 40% at a confining stress of 100 kPa. Cyclic loading was terminated at predetermined excess pore water pressure ratio levels, and the specimens were then subjected to re-consolidation to the original 100 kPa stress level and subsequent cyclic shearing. Some of the specimens were re-consolidated upon the completion of a second cyclic loading and again subjected to another round of cyclic loading. The test results indicated that the multiple cyclic loading response of loose sand depends on the degree of excess pore water pressure (or maximum shear strain) that was reached during previous loading, and the densification that occurred during re-consolidation after a previous loading.

Keywords: Multiple Cyclic Loading, Air-Pluviation, Direct Simple Shear Tests, Centrifuge Model Tests

1. INTRODUCTION

Earthquake-induced ground displacements are one of the primary hazards to structures located in seismically active areas with liquefiable soils such as those in the Fraser River Delta of British Columbia, Canada. The prediction of ground displacements using numerical models forms a critical part in seismic evaluation, design, and retrofit of structures founded on these soils. Ideally, the acceptability of numerical models requires proper validation using recorded data from field case histories. The needed field data (i.e., soil and groundwater conditions, input ground motions, displacements etc.), however, are often not available with sufficient accuracy and detail, and this, in turn, has hindered the confirmation of numerical models.

It is well known that the behaviour of soils is stress level dependent. Therefore, the use of small-scale models under natural gravity (1g) conditions, which causes stress levels that are significantly smaller than those encountered in the field, is not considered suitable to generate data for verification of numerical models. Centrifuge systems can be used to invoke a high gravitational field on small-scale soil models, thus overcoming the above stress level deficiency in 1g models, and providing an opportunity for more realistic imposition of field stress conditions. Extensive research over the past thirty five years has demonstrated the potential of centrifuge testing in examination of the response of well-defined geotechnical boundary value problems (Arulanandan and Scott, 1993; Boulanger et al., 1999; Phillips et al., 2002), and the approach has been increasingly considered as a meaningful basis for generating data for validation of numerical models.

A research program was undertaken at the University of British Columbia (UBC), Vancouver, Canada, with the aim of validating fully-coupled effective stress numerical models using data generated from centrifuge tests conducted at C-CORE (Centre for Cold Oceans Resources Engineering) research facility at Memorial University of Newfoundland, St. John’s, Canada (http://www.civil.ubc.ca/liquefaction/). The centrifuge models used in this program were constructed using dredged Fraser River sand, and the generated data from centrifuge testing provided the basis for verification of the numerical approaches as well as the modelling of some typical problems faced by the profession in relation to geotechnical earthquake engineering works in liquefiable loose deltaic sand deposits. These centrifuge models were subjected to single as well as multiple (repeated) cyclic loading.

In the modelling of a given problem, the numerical procedure should accurately capture the mechanical response of the soil. A fundamental understanding of the mechanical response can best be derived from controlled laboratory element testing of representative specimens. For these tests to be meaningful, the specimens should essentially replicate the soil conditions existent in the subject centrifuge model. Since the mechanical response of soils is well known to be dependent on its particle fabric/structure (Oda, 1972; Ladd, 1974; Mulilis et al., 1977; Vaid et al., 1999; Leroueil and Hight, 2003; Wijewickreme et al., 2005), in addition to other influencing factors such as soil type, void ratio, loading conditions, and loading history, it is critical that the anticipated particle fabric in the centrifuge specimen is also closely replicated in the specimens used for laboratory element testing.

In centrifuge testing of sand models, the physical model is commonly prepared by placing dry sand using the method of air-pluviation. For example, in the Geotechnical Centrifuge Centre at Rensselaer Polytechnic Institute (RPI), centrifuge specimens were prepared by pouring the sand from a predetermined height using a funnel with essentially the same width as the centrifuge box (Taboada and Dobry, 1993), and the centrifuge facility at the University of California, Davis (Boulanger et al., 1999) also uses the method of air-pluviation. Both the element tests conducted at UBC as well as the centrifuge tests conducted at C-CORE comprise of Fraser Sand prepared using the method of air-pluviation. Any required dense zones in the centrifuge model were achieved by tamping after the process of air-pluviation.

The cyclic resistance to liquefaction has also been noted to be significantly influenced by past liquefaction, or pre-shearing, effects (Finn et al., 1970; Ishihara and Okada, 1978, 1982; Suzuki and Toki, 1984; Vaid et al., 1989). Finn et al. (1970) noted this effect based on the results obtained from direct simple shear (DSS) and triaxial tests conducted on water-pluviated sand specimens. They found that previously liquefied specimens exhibit significantly less cyclic shear resistance than virgin specimens despite a significant increase in density due to consolidation following liquefaction. The specimens that did not reach liquefaction on first loading, and were allowed to consolidate, showed significant increase in their resistance to liquefaction in comparison to those observed for virgin specimen. Ishihara and Okada (1978, 1982) have distinguished between the small and large pre-shearing by the location of the effective stress state with respect to the "line of phase transformation". Ishihara and Okada (1978) and Vaid et al. (1989) indicated that the small pre-shearing would significantly reduce the excess pore water pressure generation during subsequent cyclic loadings. On the other hand, large pre-shearing could significantly increase or decrease the pore pressure generation in next loading depending on the loading direction (Vaid et al., 1989). If a specimen is loaded in the same direction as the direction of pre-shearing (i.e., no strain reversal) then the pore pressure generation was noted to be less than that observed during the previous loading, and vice versa. The observations made by Ishihara and Okada (1978, 1982) and Vaid et al. (1989) were based on the results obtained from triaxial tests conducted on water-pluviated sand specimens. Suzuki and Toki (1984) also observed similar responses in triaxial tests conducted on air-pluviated sand specimens.

While some databases are available for element tests conducted on water-pluviated specimens to assess the effects of past liquefaction or pre-shearing, laboratory testing on air-pluviated sands is scarce specially under DSS loading conditions. Centrifuge specimens at C-CORE centrifuge facility were prepared using air-pluviated sand, and the cyclic loading in centrifuge is considered to be more effectively replicated in a simple shear device rather than a triaxial device. Therefore, there is a need to obtain data from element tests using the DSS device on air-pluviated specimens that closely mimic the soil fabric and stress conditions of the centrifuge specimens. These tests are required for the validation of numerical models using centrifuge tests that are subjected to multiple cyclic loading. In recognition of this, a detailed laboratory element testing research program was undertaken focusing on the multiple cyclic shear response of loose air-pluviated Fraser River sand.

The NGI-type (Bjerrum and Landva, 1966) DSS device at UBC was used to carry out the testing. Unlike laboratory studies conducted on specimens prepared using now well-established methods of water-pluviation, it was recognized that the re-constitution of soil specimens to mimic the particle fabric in the centrifuge physical soil models is non-routine. Since this is an important prerequisite as well as an integral component of this study, great importance was given to the development of pluviation techniques for specimen re-constitution and associated verification of specimen quality (Wijewickreme et al., 2005).

2. MATERIAL TESTED AND EXPERIMENTAL DETAILS

The selection of soil material and the development of specimen preparation to characterize the multiple cyclic loading response of loose air-pluviated sand were undertaken to directly support and complement the numerical modelling and centrifuge testing research work conducted by UBC and C-CORE.

2.1. Material Tested

The Fraser River sand used in this study had an average particle size D_{50} = 0.26 mm, D_{10} = 0.17 mm, and uniformity coefficient c_u = 1.6. This dredged sand from the Fraser River in the Lower Mainland of British Columbia, Canada, has been extensively used in laboratory research at UBC over the past 20 years. The maximum and minimum void ratios (e_{max} and e_{min}) for the sand determined as per American Society for Testing and Materials Standards ASTM-4254 and ASTM-4253 are 0.94 and 0.62, respectively. Fraser river sand is composed of 40% quartz, quartzite, and chert, 11% feldspar, and 45% unstable rock fragments (Garrison et al., 1969). The sand grains are generally angular to sub-rounded in shape. The grain size distribution and microscopic view of the Fraser River sand particles are shown on Fig. 1.

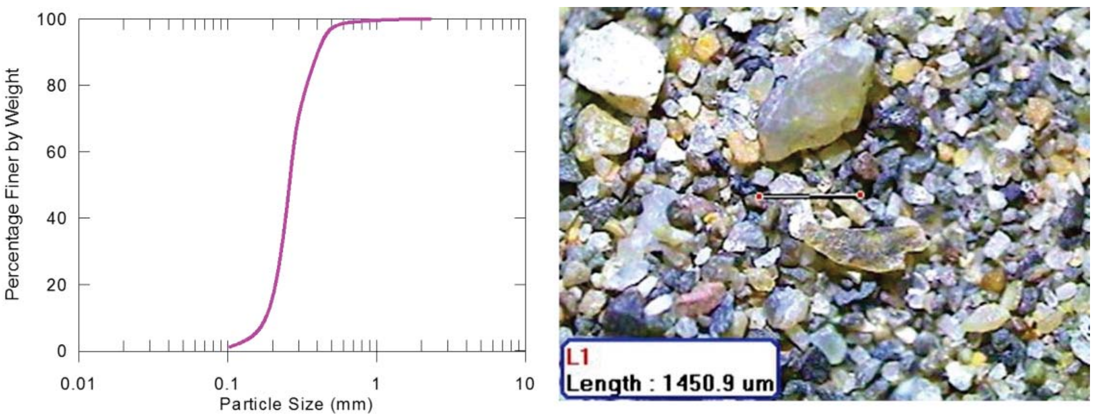

Figure 1. Grain size distribution and microscopic view of Frazer River sand.

2.2. Experimental Details

The NGI-type DSS device at UBC allows the testing of a specimen having a diameter of 70 mm and height of 20 to 25 mm, under constant-volume condition. In constant-volume DSS tests, as an alternative to preventing drainage of a saturated specimen, a constant volume condition can be enforced even in a dry soil by constraining the specimen boundaries (i.e., diameter and height) against changes. The specimen diameter is constrained against lateral strain using a steel-wire reinforced rubber membrane, and the height constraint is obtained by clamping the vertical movement of the top and bottom loading caps. It has been shown that the decrease (or increase) of vertical stress in a constant-volume DSS test is essentially equal to the increase (or decrease) of excess pore water pressure in an undrained DSS test where the near constant volume condition is maintained by not allowing the mass of pore water to change (Finn et al., 1978; Dyvik et al., 1987; Wijewickreme et al., 2005).

In order to investigate the element behaviour under multiple cyclic loading conditions, a series of constant-volume cyclic DSS tests were conducted on dry air-pluviated Fraser River sand specimens, as outlined in Table 2.1. These tests were conducted at a vertical confining stress (σ'_{vc}) level of 100 kPa with no static shear stress bias. All the specimens were air-pluviated to an as-placed relative density of about 34% with the intent of achieving a target relative density at the end of consolidation (D_{rc}) of about 40% at a vertical confining stress of 100 kPa. This σ'_{vc} - D_{rc} combination was specifically chosen since it represented the target loose density of the C-CORE centrifuge models.

Table 2.1. Summary of multiple cyclic loading constant-volume DSS tests

Specimen No.	Initial Relative Density, D_{rc} (%)	Cyclic Loading	Cyclic Loading Phase No.	Excess Pore Water Pressure Ratio (r_u) at which Cyclic Loading was terminated (%)
R1	41	1st	R11	100
		2nd	R12	100
R2	41	1st	R21	46
		2nd	R22	100
R3	41	1st	R31	88
		2nd	R32	100
R4	40	1st	R41	100
		2nd	R42	100
		3rd	R43	100
R5	40	1st	R51	54
		2nd	R52	100
		3rd	R53	100

In these stress-controlled constant-volume cyclic tests, the cyclic loading (τ_{cyc}/σ'_{vc} = 0.1) was applied in the form of a sinusoidal wave with a frequency of 0.1 Hz. Even though this frequency is less than the frequency content of typical earthquake loadings, it enabled a better control of loading as well as data acquisition. The undrained behaviour of sand is known to be essentially frequency independent; therefore, this approach is commonly adopted as reasonable in laboratory cyclic loading of granular soils.

The first cyclic loading was stopped at predetermined excess pore water pressure ratio (r_u) levels, and the specimens were then subjected to re-consolidation to the original 100 kPa stress level and subsequent cyclic shearing. Some of the specimens were re-consolidated upon the completion of a second cyclic loading and again subjected to another round of cyclic loading (Table 2.1). All these specimens were manually reset in a strain-controlled manner to reach approximately zero shear stress and strain level before subjecting to re-consolidation.

The air-pluviation technique used to re-constitute the Fraser River sand and associated verification of specimen quantity are presented in detail in Wijewickreme et al. (2005), and therefore, not repeated herein.

3. RESULTS AND DISCUSSIONS

Figs. 2 to 4 present the stress paths and stress-strain responses under first and second cyclic loading phases of Specimen R1 through R3, respectively. As indicated in Table 2.1, the specimens had identical initial conditions (an initial relative density of 41% at a confining stress level of 100 kPa) and the first cyclic loading of a given specimen was stopped upon reaching a certain predetermined r_u and allowed to consolidate. The specimens were then subjected to a repeated cyclic loading.

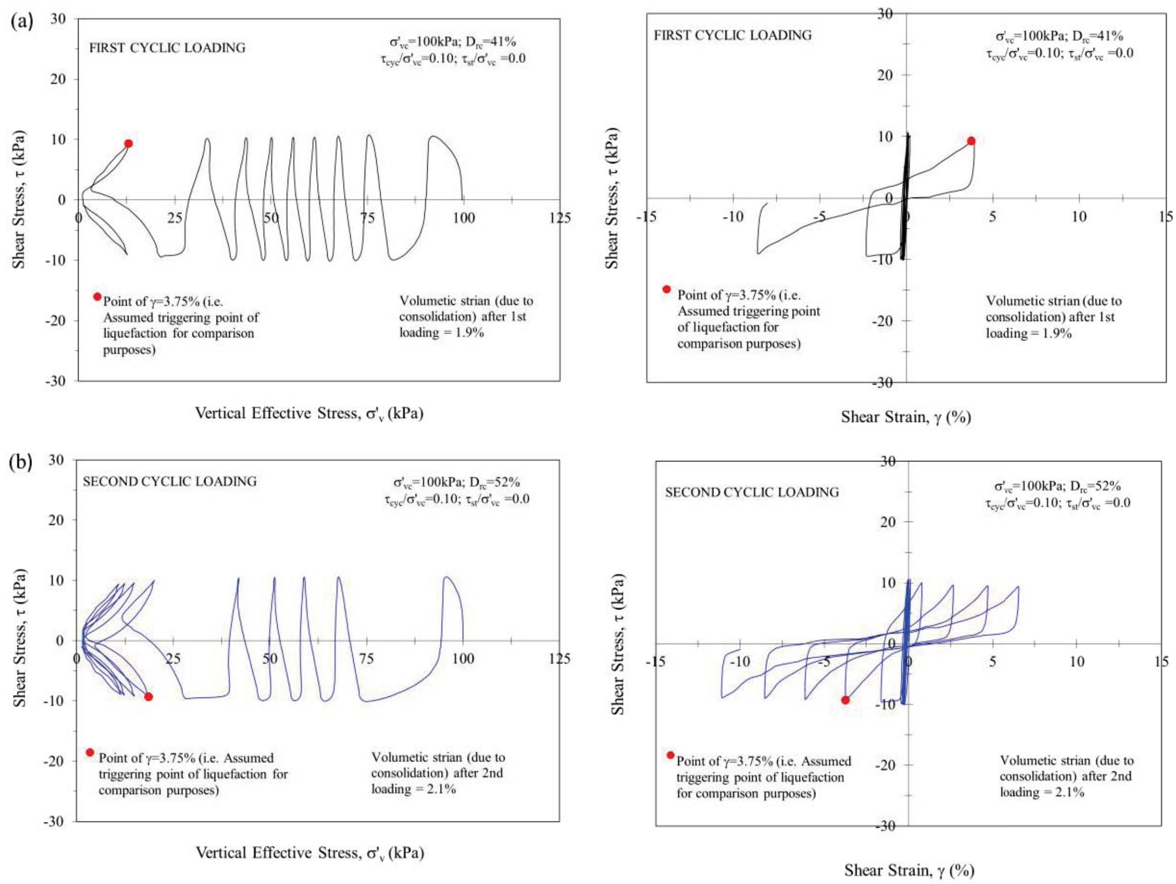

Figure 2. Constant-Volume DSS Test Results – Specimen R1

Specimen R1 was subjected to first cyclic loading (Phase R11) until it reached a r_u of 100%. Specimen R1 showed a significant drop in σ'_{vc} with increasing number of cycles. Specimen R1 reached single-amplitude horizontal shear strain (γ) of 3.75% in the 9th cycle. (Note: The number of load cycles required to reach $\gamma = 3.75\%$, in a given constant-volume DSS test under a given applied Cyclic Stress Ratio (CSR), is essentially equivalent to reaching a 2.5% single-amplitude axial strain in a triaxial soil specimen. An identical definition has been previously used to assess the cyclic shear resistance of sands by the US National Research Council (NRC, 1985), and it also has been adopted in many previous liquefaction studies at UBC).

During re-consolidation, the specimen experienced a volumetric strain of 1.9%, and this resulted in an increase of relative density to a value of 52%. As shown on Fig. 2(b), in the second cyclic loading phase (R12), this specimen reached $\gamma = 3.75\%$ after reaching lesser number of cycles (6 cycles) than first cyclic loading (R11) despite the increase in the relative density. This indicates that the high excess pore water pressures (r_u = 100%) incurred during Phase R11 resulted in weakening the soil. The observed behaviour is in accord with previous findings by others on the response of water-pluviated specimens subjected to large pre-shearing (Finn et al., 1970; Ishihara and Okada, 1978; Vaid et al., 1989).

Specimen R2 was subjected to first cyclic loading phase (R21) until it reached a r_u of 46% (Fig. 3(a)). This r_u value was reached in 5 cycles of loading and only with very small amplitude of shear strain (γ_{max} = 0.18%). The noted good agreement between the first five cycles of loading in this loading phase (R21) and the Phase R11 of previous specimen confirms the repeatability of the test results. A volumetric strain of 0.2% and a relative density increase to a value of 42% was noted during re-consolidation. In the second cyclic loading phase (R22), this specimen reached $\gamma = 3.75\%$ after reaching significantly larger number of cycles (28 cycles) than first cyclic loading (Phase R11). This noted significant increase in cyclic shear resistance cannot be due to the small increase in the relative density from 41% to 42%. This indicates that some strengthening in the soil fabric that may have taken place during Phase R21 that was terminated before generating larger r_u values (r_u = 46%) and shear strain (γ_{max} = 0.18%). This observed behaviour is also in accord with the previous findings by others on the response of water-pluviated specimens subjected to small pre-shearing (Finn et al., 1970; Ishihara and Okada, 1978; Vaid et al., 1989).

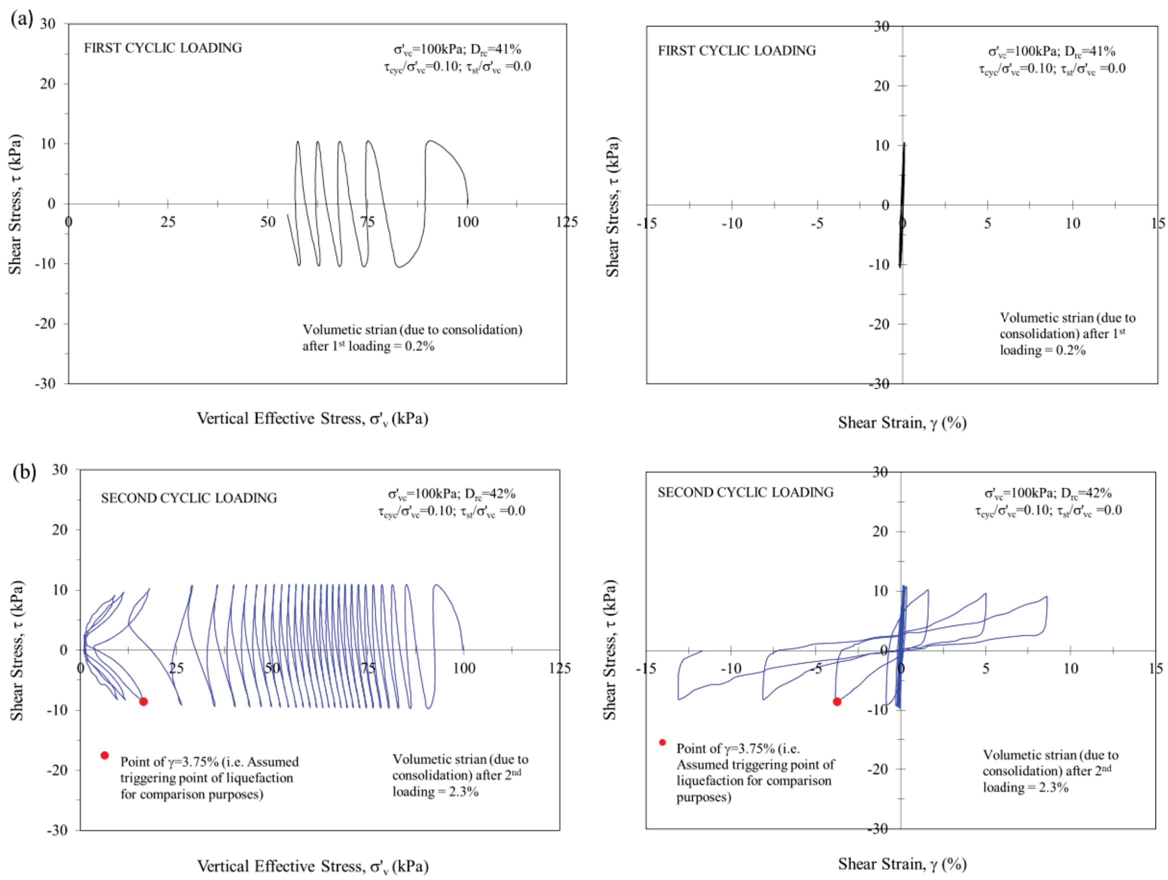

Figure 3. Constant-Volume DSS Test Results – Specimen R2

Specimen R3 was subjected to first cyclic loading (Phase R31) until a r_u of 88% that was reached during the 8th cycle (Fig. 4(a)). A maximum shear strain of 2.1% was experienced by the specimen during this stage. While this strain level is significantly less than that experienced by the Specimen R1 in Phase 11 (γ_{max} = 8.0%), it is still lager than that occurred in Phase R21 of Specimen R2 (γ_{max} = 0.18%). There is good agreement in the response between this first cyclic loading phase and those observed from Phases R11 and R21, again, confirming the repeatability of the test results. The Specimen R3 was then re-consolidated to 100 kPa confining stress level as before. A volumetric strain of 0.8% was noted during re-consolidation and the relative density of the specimen was increased to 46% during this process. As shown on Fig. 4(b), in the second cyclic loading phase (R32), this specimen reached $\gamma = 3.75\%$ after reaching lager number of cycles (27 cycles) than that required for $\gamma = 3.75\%$ in first cyclic loading phase (R11). It is of interest to note that during the second cyclic loading Specimen R3 reached $\gamma = 3.75\%$ after reaching almost the same number of cycles as that observed from Phase R22 despite the difference in the relative densities prior to second loading.

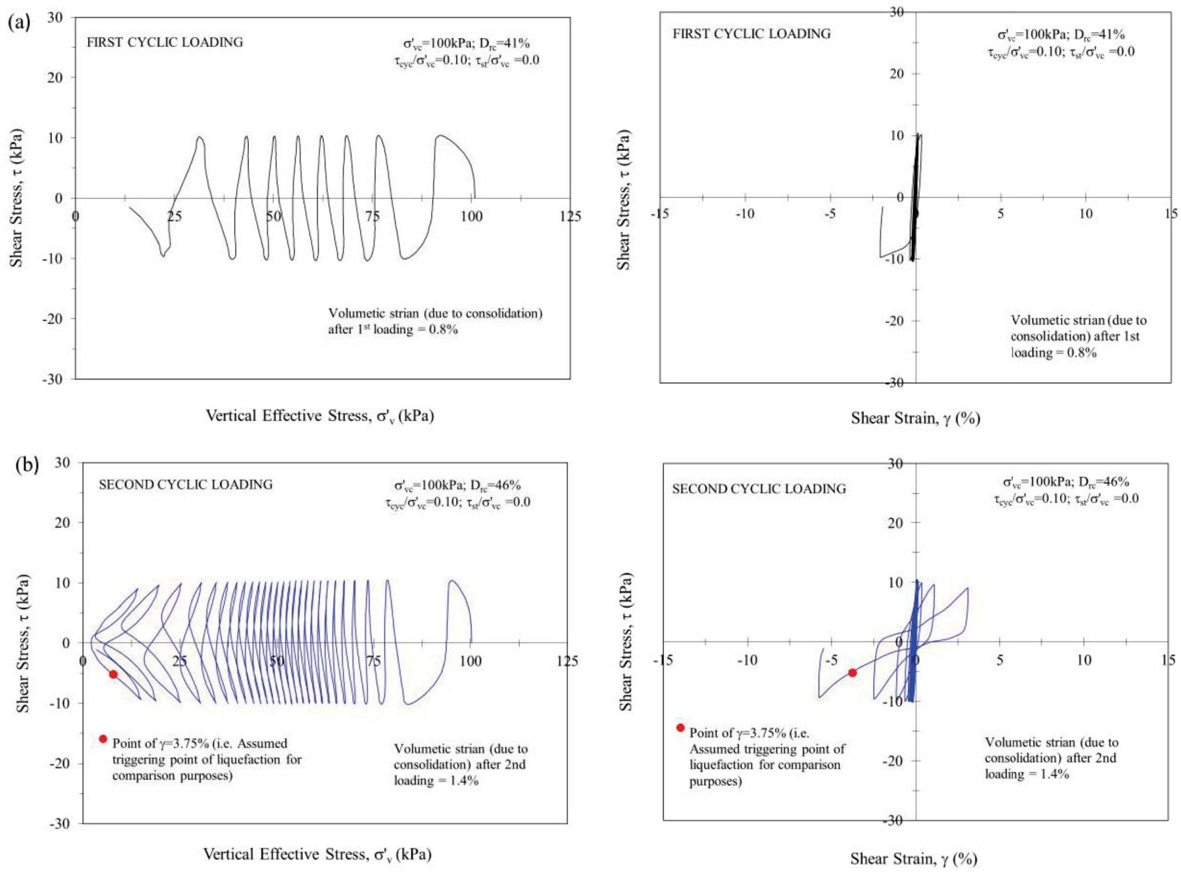

Figure 4. Constant-Volume DSS Test Results – Specimen R3

Figs. 5 and 6 present the stress paths and stress-strain responses under first, second, and third cyclic loading phases of test numbers R4 and R5, respectively. As indicated in Table 2.1, these specimens had identical conditions (an initial relative density of 40% at a confining stress level of 100 kPa) and the first and second cyclic loadings of a given specimen were stopped upon reaching a certain predetermined excess pore water pressure ratio (r_u).

Specimen R4 was subjected to first cyclic loading (phase R41) until it reached a r_u of 100% and reached $\gamma = 3.75\%$ in the 7^{th} cycle. During re-consolidation in preparation for loading Phase R42, the specimen experienced a volumetric strain of 2.0 % and a relative density increase to a value of 51.5%. In the second cyclic loading phase (R42), this specimen reached $\gamma = 3.75\%$ after reaching almost same number of cycles (8 cycles) as Phase R41 despite the larger relative density.

It is noted that the Specimen R4 and loading Phase R41 are essentially identical to the Specimen R1 and its first loading Phase R11. As such, the observed responses in second loading phase for both the specimens are very similar. Once again, this almost unchanged cyclic shear resistance suggests a clear weakening in the soil fabric due to the first occurrence of r_u = 100%. When the Phase R42 was terminated the specimen had reached a r_u of 100%.

The specimen was, again re-consolidated to a vertical confining stress of 100 kPa to prepare for the next cyclic loading phase (R43). During this consolidation, the specimen suffered an additional volumetric strain of 1.2% and the relative density was increased to 58.4%. As shown on Fig. 5(c), during third cyclic loading Phase (R43), this specimen reached $\gamma = 3.75\%$ in the 17^{th} cycle, which is higher than the required number of cycles to cause $\gamma = 3.75\%$ during first two cyclic loading phases (R41 and R42). This increase in the cyclic resistance during third cyclic loading appears to be arising from increase in density that took place during re-consolidation after Phase R42. Based on the previous observed trends, since the specimen R4 reached r_u = 100% in Phase R42, it should have exhibited significant weakening in Phase R43. It appears that any such possible degradation of cyclic strength has been over-shadowed by the increase in the relative density.

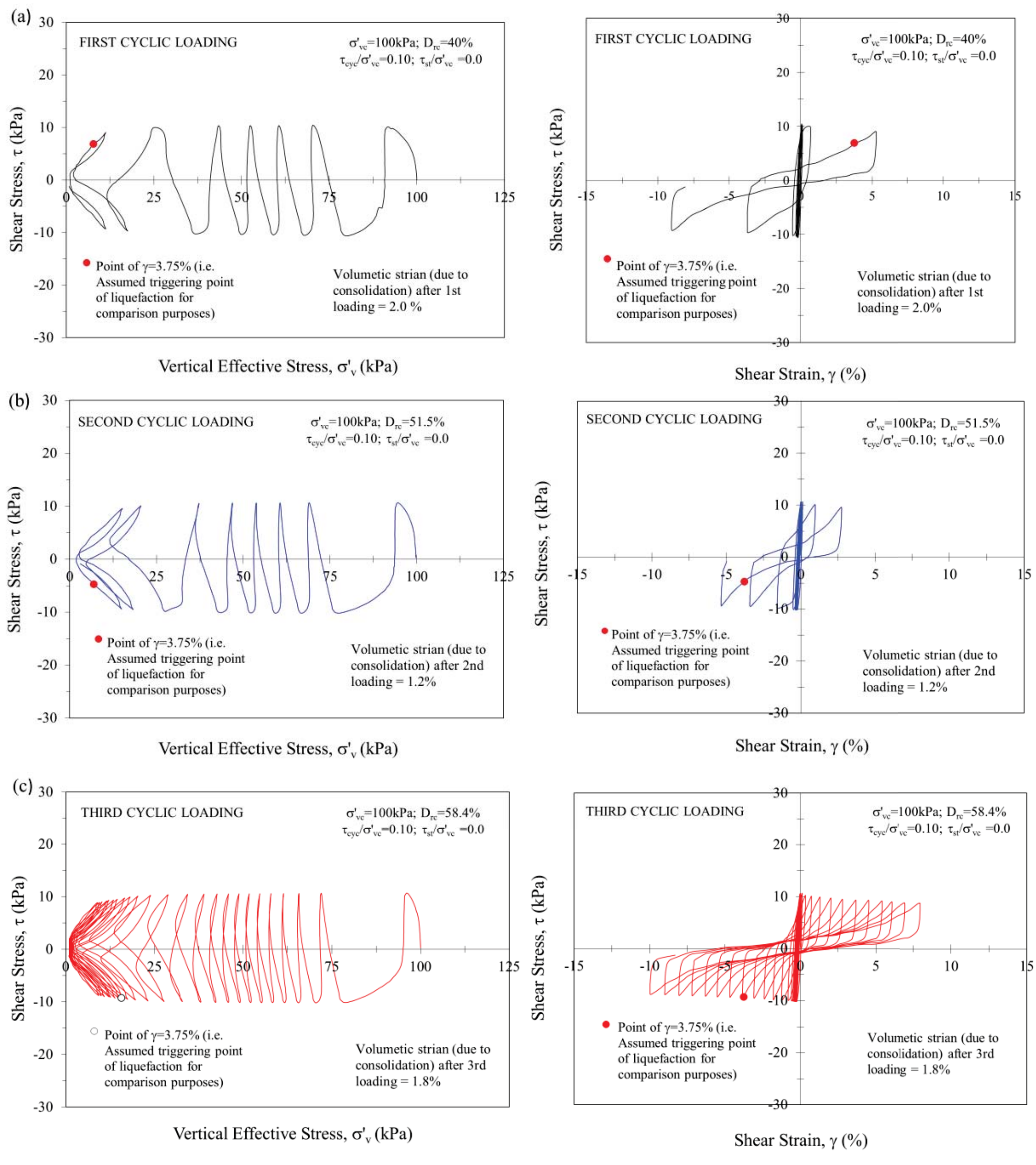

Figure 5. Constant-Volume DSS Test Results – Specimen R4

Specimen R5 was subjected to loading Phase R51 until it reached a r_u of 54% after 5 cycles of loading. A volumetric strain of 0.2% (with D_r reaching to a value of 41%) was noted during re-consolidation for Phase R52. The loading Phase R52 required significantly larger number of cycles (25 cycles) to reach γ = 3.75% (Fig. 6(b)). This is essentially identical to Phase R22 (Fig. 3(b)) and the observations can be explained using potential strengthening in the soil fabric due to small pre-shearing as before. During second re-consolidation, the specimen experienced a volumetric strain of 2.2% and the relative density was increased to 54%. As shown on Fig. 6(c), in the third cyclic loading Phase R53, this specimen reached $\gamma = 3.75\%$ in the 8th cycle, which is almost same as the required number of cycles to reach $\gamma = 3.75\%$ during first cyclic loading Phase R51. Herein, any strengthening in soil fabric associated with small pre-shearing appears to be erased due to fabric alterations due to the occurrence of $r_u = 100\%$ during Phase R52.

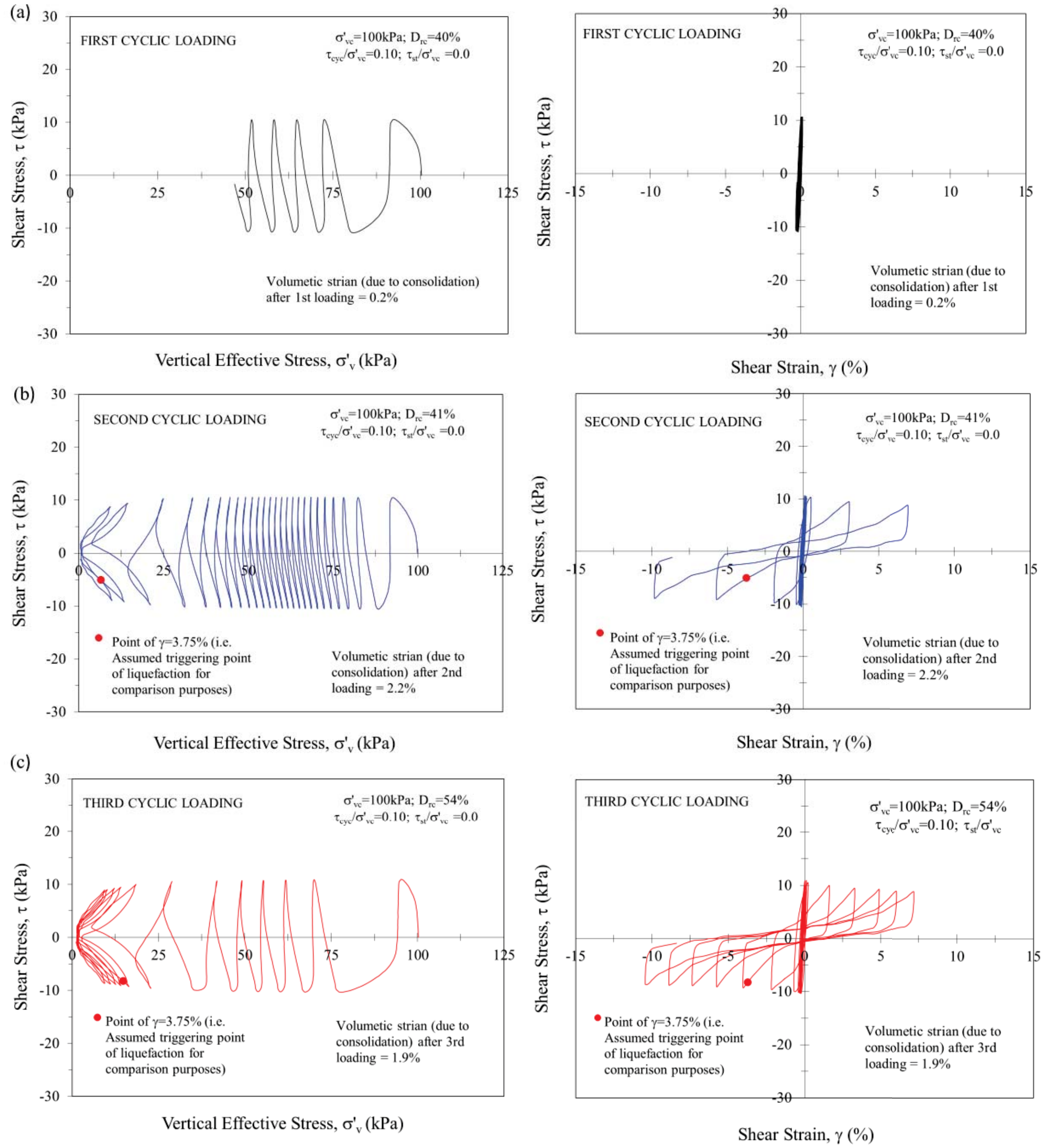

Figure 6. Constant-Volume DSS Test Results – Specimen R5

4. CONCLUSIONS

Constant-volume cyclic DSS tests conducted on air-pluviated loose Fraser River sand specimens that had been previously subjected to cyclic loading indicate that the response of sand depends on the degree of excess pore water pressure (or maximum shear strain) that was reached during previous cyclic loading, as well as the densification that occurred during re-consolidation after previous cyclic loading. The following were noted in particular:

- Cyclic shearing that did not impart significant excess pore water pressure or shear strains increases the resistance of sand against liquefaction in future cyclic loadings;
- Relatively large levels of excess pore water pressure or shear strains during cyclic loading weaken the soil fabric and, in turn, decrease the resistance of sand against liquefaction in future cyclic loadings; and
- The reduction in the cyclic resistance of a given sand to liquefaction due to large cyclic pre-shearing may, however, be overridden by the increase in relative density that take place during the reconsolidation process.

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