Measurement of Hydraulic Conductivity Variation in Post Seismic Behavior of Sand Using TxSS

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ABSTRACT
Many efforts have been invested in the establishment of pore pressure models and evaluation of the influence factors in the seismic behavior of a saturated soil. However, no thorough studies took into consideration the transformation of the physical soil properties and the impact of the significant parameters in the post-seismic phase. The variation of permeability throughout the liquefaction and post liquefaction phase was more tackled by exhaustive empirical methods to produce numerical models. Conversely, in this paper, the post-seismic behavior of soil is triggered by an experimental manner using the Simple Triaxial Shear Test (TxSS). Different analyses were drawn after performing several undrained cyclic strain-controlled and permeability tests (using TxSS) on Ottawa sand. It is proved, that the hydraulic conductivity variation can be calculated during the excess pore pressure dissipation time history by using a simple evaluation method. The comparison of the results obtained from TxSS and conventional permeameter, validates the use of TxSS instead of the conventional test especially in determining the hydraulic conductivity variation.

RÉSUMÉ
De nombreux efforts ont été déployés pour établir des modèles de développement de pression interstitielle et pour évaluer les facteurs d'influence du comportement sismique d'un sol saturé. Cependant, aucune étude approfondie n'a pris en compte l'évolution des propriétés physiques du sol et l'impact des paramètres significatifs dans la phase post-sismique. La variation de la perméabilité tout au long de la phase de liquéfaction et de post-liquéfaction a été davantage abordée par des méthodes empiriques exhaustives pour élaborer des modèles numériques. Contrairement à cet article, où le comportement post-sismique du sol est approché expérimentalement en utilisant le test de cisaillement triaxial simple (TxSS). Différentes analyses de plusieurs tests cyclique non drainée et de perméabilité (en utilisant TxSS) ont été effectuées sur du sable d'Ottawa. Il a été prouvé que la variation de la conductivité hydraulique peut être calculée pendant la dissipation de la pression interstitielle en excès en utilisant une méthode d'évaluation simple. La comparaison des résultats obtenus à partir de TxSS et du perméamètre conventionnel valide l'utilisation de TxSS au lieu du test conventionnel en particulier pour déterminer la variation de conductivité hydraulique.

1 INTRODUCTION

Liquefaction analysis took several trends of evaluations, some involve experimental testing and other were based on numerical modeling. The evaluation of liquefied loose saturated soils that experienced a seismic activity has been mainly focused on one main goal. This goal is established in analyzing the related increase of excess pore pressure and the accompanied loss of soil resistance (Seed and Idriss 1971, Dobry et al. 1982, Green et al. 2000). In fact, understanding the stress path of the liquefied soil was an important step in such evaluation, however, many alteration in the state intrinsic parameters should be taken into account to better resemble the phenomenon, especially when dealing with numerical modeling. Permeability is a key soil parameter, which increases during liquefaction and decreases reaching its initial value or less after dissipation of excess pore pressure. Such an alteration was considered in evaluating a pore pressure dissipation model for post liquefaction assessment (Ha et al. 2003, Shahir and Pak 2010, Ghasemi and Pak 2011, Haigh et al. 2012, Rahmani and Pak 2012, Shahir et al. 2012 and Wang et al. 2013). Several studies triggered the difference between the adoptions of a constant initial permeability coefficient, increased permeability coefficient and the variation of permeability in addressing the liquefaction analysis numerically. The last latter was able to capture approximately all features of soil response. A good prediction of pore pressure generation and dissipation as well as a good agreement of ground deformation results were shown in a model which is based on the variation of permeability coefficient (Rahmani et al. 2012).

On the other hand, many efforts have been made to experimentally measure permeability before, during and after liquefaction in an attempt to study the variation of this parameter as well as creating a relation with the controlling factors in post-liquefaction phase. In this context, the volume change had and still been tackled as a first stage. Limited researches conducted identical experimental setup for the sake of permeability and determination of other parameters measurements during liquefaction (Yoshimi et al. 1975, Haigh et al. 2012, Ueng et al. 2017). However, they followed an experimental setup which consists of a tall cylindrical soil sample of approximately a height to diameter ratio ranges from 2 to 5. The mechanism is relied on subjecting the soil column to vibration with and without flow of water. This is a good approach for predicting the variation of permeability. Nevertheless, the question remains whether these are well representative for a liquefaction study compared to specimen's height to diameter ratio is of a range 0.32 to 0.46 and which is subjected to confining pressure prior to simple shearing by a non-uniform waveform using the Triaxial Simple Shear Test (TxSS). In this paper, undrained cyclic tests were performed on Ottawa Sand C-109 using the TxSS. The generation and dissipation of pore pressure were evaluated. The results show the dependence of sand post cyclic behavior on the dissipated energy as well as the cyclic strain. On the other side, in the attempt to understand the relation between dissipated energy, cyclic strain and permeability, previously the authors in Bayoumi et al. 2017 discussed the possibility of attaining a direct measurement of permeability variation throughout the dissipation process. This paper elaborates this issues by considering a simple hydraulic conductivity measurement approach based on flow rate and dissipated pressure.

2 EXPERIMENTAL SETUP

The ability to rotate principle stresses, ability to simulate the three dimensional in situ conditions of the soil and the ability to apply confining pressure to the soil samples are all ensured with minimum impact in the combined triaxial simple shear (T_xSS) (Chekired et al. 2015). Cyclic triaxial simple shear test (TxSS) apparatus, used in this study, was designed to provide confining pressure to the soil specimen during simple shearing by a non-uniform waveform. Comparing this test to other cyclic tests, TxSS will give direct measuring of pore pressure rather than pressure deduction method which leads to inaccurate results most of the time. Due to some limitations of the testing apparatuses used to evaluate the static and dynamic responses for the cohesion and cohesionless soils in liquefaction analysis, the efficiency of the TxSS can verify the significance consideration of some aspects which are avoided in other testing apparatuses.
Cyclic undrained testing at a certain specified conditions mentioned in the next section is performed using the TxSS to shear the soil specimen and evaluate its resistance and accompanied increase in excess pore pressure. Dissipation of excess pore pressure (post-liquefaction behavior) is permitted in a drained testing after the cyclic test and the volume change is measured via a volume change apparatus. The flowrate of an induced water flow of approximately 3 to 4 ml, prior to undrained cyclic testing and after drained testing, is measured in order to calculate the Pre-cyclic and Post-dissipation hydraulic conductivity respectively.

3 EXPERIMENTAL WORK

3.1 Specimen Preparation and Experimental Methodology

The uniform Ottawa C109 sand with the grain size distribution curve shown in Figure 1 and the properties mentioned in Table 1 is used in this study. Wet tamped

Table 1. Parameters of tested soils

Parameters	Ottawa C-109
D_{50} (mm)	0.38
Cu	1.83
Cc	1.06
e_{min}	0.48
e_{max}	0.82
Gs	2.65

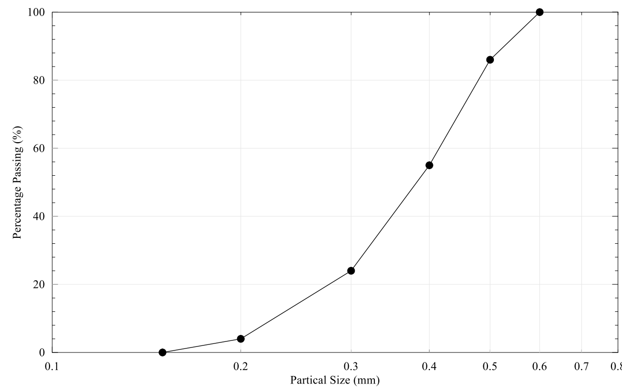

Figure 1. Grain size distribution curve of Ottawa C-109 Sand

preparation method was used to prepare reconstituted soil specimens in unreinforced rubber membrane where the specimen is set between porous cap and base. The soil specimen is of 79 mm in diameter and 37 mm height. Moist sand was placed in three layers and every layer was compacted to reach the desired density. All samples prepared for testing were of a dry density 1500 Kg/m^3 which is equivalent to loose state of relative density equals to 15%. After saturation, with a Skempton's B value greater or equal to 0.97, the sample was isotropically consolidated to an initial stress ratio of $k_o = (\sigma'_h/\sigma'_v) = 1$, where σ'_v and σ'_h are the effective vertical and horizontal stresses respectively (Chekired et al. 2015). Soil samples of a diameter (79 mm) to height (37 mm) ratio (D/H) equals to 2.14 have been carried out at the initial effective confining pressure of 100 kPa. The D/H is not as requested by ASTM; however, the increase of the height was necessary to have a better hydraulic conductivity measurement. A minimum diameter of 71 mm and a D/H ratio of 1 to 2 (Carpenter G., et al. 1986) can give a good prediction of hydraulic conductivity. Further discussion about D/H restrictions in cyclic loading in TxSS is shown in section 4.2. The samples are subjected to the same loading frequency of 1 Hz. A series of undrained cyclic strain-controlled tests under isotropic stress conditions and at different values of cyclic strain γ_{cyc} are conducted until initial liquefaction occurs. After liquefaction takes place, drainage is permitted and the volume change is recorded via a volume change apparatus to measure the dissipated volume. In addition to excess pore pressure generation and dissipation are recorded throughout both cyclic and dissipation processes.

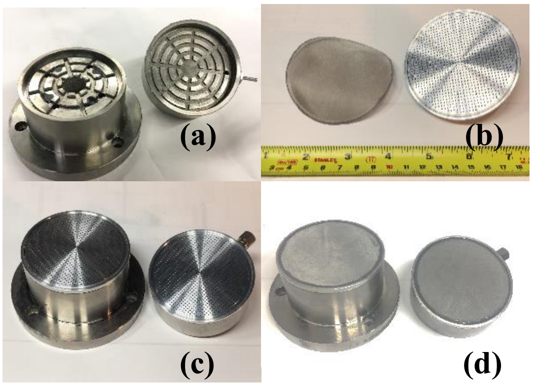

Figure 2. Manufactured porous stones with attached sieve mesh

3.2 Hydraulic Conductivity Measurement and Concerns

In order to investigate the hydraulic conductivity change comparing the initial and final one, a set of 3 to 4 ml water flows under different constant pressure heads 2, 5 and 10 kPa are performed. Controlling the flow seems to be more preferable and accurate rather than controlling the flow time as required by ASTM in the permeability flexible wall. Hydraulic conductivity assessment is performed for four testing samples as shown in Table 3. As a result, four cyclic tests of different cyclic strain, same relative density (35% to 40%) and same diameter to length ratio equals to 2.14 are compared concerning dissipated energy, Du, dissipated volume and flowrate. It is important to mention that usually all permeability tests regardless of the apparatus used are performed by placing the soil sample between two porous stones. Several researches warned about this influence of the porous stone in adjusting the hydraulic conductivity of the soil itself. As a consequence, some used "Eq. 1" to calculate the hydraulic conductivity with less errors possible considering that the hydraulic conductivity measured is the answer of the whole system which is constituted of porous stones and soil together.

$$K_{soil} = \frac{L_{soil}}{\frac{L_{sys}}{K_{sys}} - \left(\frac{L_t}{K_t} + \frac{L_b}{K_b}\right)} \qquad [1]$$

Where: K_{soil} is the saturated hydraulic conductivity of the soil sample. K_{sys} is the measured value of hydraulic conductivity of the whole system (soil with top and bottom porous stones). L_{soil} and L_{sys} is the length of the soil sample and the whole system respectively. L_t and L_b is the length of the top and bottom porous stones respectively. K_t and K_b is the hydraulic conductivity of the top and bottom porous stones respectively. However, the use of the above equation can not be efficient to eliminate the influence of the porous stone if there hydraulic conductivity is less than that of the soil. As a consequence, according to several permeability tests performed, the authors realized that in order to exclude the effect of the porous stones, manufactured aluminum porous stones are made of a pore size of 0.7 mm and a sieve of 25 mesh size is attached to it to prevent infiltration and clogging of the particles in the pores as shown in Figure 1. After several testing on various

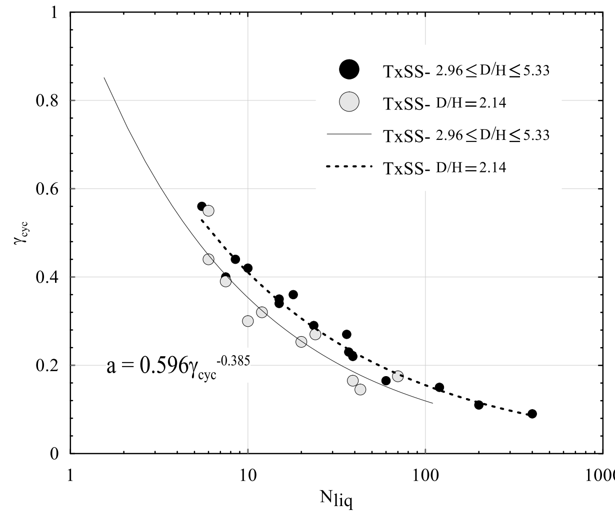

Figure 3. Results of γ_{cyc}–N_{liq} varaiation for different D/H ratios

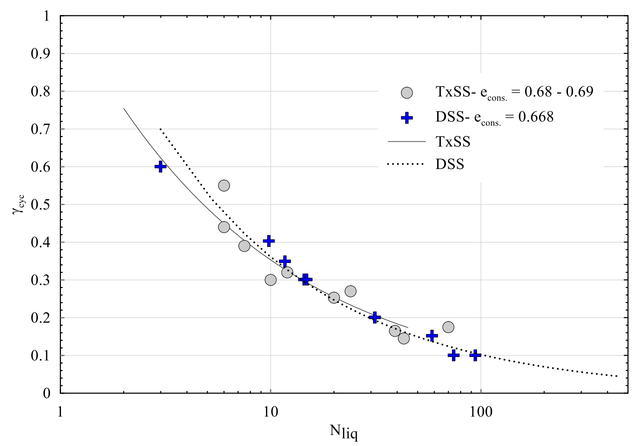

Figure 4. Results of γ_{cyc}–N_{liq} varaiation using TxSS and DSS (Sibley et al., 2017) testing apparatuses

types of soils, the manufactured porous stones allows the persistence of accurate soil hydraulic conductivity.

3.3 Diameter to Height Ratio Effect

The specimen size is considered one of the important concerns in cyclic testing. The non-uniformity of shear stress distribution, which is specifically explained by the deviation of shear stresses at the boundaries compared to that at the middle, should be one of the avoidable issues.
Several studies about D/H ratio effect on cyclic testing are found in literature and they most tackle the DSS (Budhu 1983, Amer et al. 1986, Kovacs and Leo 1981, Park and Jeong 2013…). The restriction of D/H ratio is quietly understood for maintaining uniform stress distribution especially by relatively decreasing the height and increasing the diameter. Apparently, the vertical constraints and boundary effects in DSS can explain this high concern. However, the case may change in TxSS, where the sample is subjected to confining stresses. Figure 3, shows a small experimental investigation concerning the

influence of D/H in TxSS. The relative density and diameter (79 mm) are maintained for all tests shown in Figure 3, whereas the height is varied. All testing specimens that there D/H is within the range 2.96 and 5.33 follows the same trend in γ_{cyc}–N_{liq} variation; where the increase of cyclic amplitude leads to a decrease in the number needed to liquefy a sample. Nevertheless, beyond 2.96 the trend is shifted downwards, which relatively confirms the decrease in soil resistance for high samples. In this study, D/H is taken as 2.14 because relatively a ratio between 1 and 2 is seems applicable in hydraulic conductivity measurement due to the slight decrease in void ratio when the height decreases. On the other hand, this slight difference in liquefaction analysis for high samples wouldn't seem inconvenient, especially when the results of specimens with D/H=2.14 according to Figure 4 are compatible with those of DSS (Sibley et al., 2017). It is important to note that the soil specimens that are tested by either DSS or TxSS shown in Figure 4 are subjected to same preparation and loading conditions except for D/H ratio. In addition, the downward shift proves that there is no over estimation of the resistance. This paper is focused on the post-liquefaction hydraulic conductivity measurement. Consequently, it is enough to attain representative liquefied state of a sample in order to perform the necessary post hydraulic conductivity assessment.

4 EXPERIMENTAL RESULTS AND ANALYSIS

4.1 Excess Pore Pressure Generation and Dissipation

An example of the build up pressure during the cyclic test on Ottawa sand under a cyclic shear strain of γ_{cyc}=0.27 is shown in Figure 7. Initial liquefaction is defined throughout this study as the excess pore pressure, $R_u=u/\sigma'_{co}$ of 0.9 to 1; where u is the residual pore water pressure generated and σ'_{co} is the initial effective confining pressure. It is well demonstrated that liquefaction is reached in sample of relative density 39% of Ottawa Sand after approximately 24 cycles. Figure 7 reveals the variation of CSR (Cyclic stress ratio) as a function of the distortion of cyclic strain in the form of a hysteresis loops. Where the dissipated energy per unit volume is calculated by integrating area bound by stress– strain hysteresis loops as suggested by Green et al. 2000. The compatibility of the experimental test performed based on earthquake (sig 4) and the theoretical model is shown in figure 7. After the cyclic test, the dissipation of pore pressure is permitted and measured along with the volume change and flowrate. The beginning of post-liquefaction is identified by a $\Delta u= u/\sigma'_{co}$ of 0.9 or 1. The increase of the flowrate and dissipated volume as a function of Δu decay for the different four tests are shown in Figure 5. As presented in table 2, a difference in cyclic amplitude can lead to difference in post-seismic behavior of soil. When a sample is loaded with a high cyclic strain, less energy is needed to liquefy a sample and as a result less cycles are required for liquefaction to occur (see test 4). In fact, at same relative density, volumetric strain increases as cyclic amplitude increases. However, the results vary slightly when comparing test 2 to test 3, regardless of the higher cyclic strain. Test 3 gives slightly less volumetric strain compared to test 2. This can be explained due to maintaining same value of dissipated energy. The dissipated energy decreases gradually as the cyclic strain amplitude increases reaching a constant value, in which beyond this value the energy remains constant. The flowrate, which linearly decreases as the pore pressure decreases, reveals the decrease in the hydraulic conductivity when dissipation takes place. At the end of the test, the flowrate variation with Δu shows a nonlinearity due to zero excess pore pressure stabilization which leads to a volume change stabilization.

Table 2. Performed Tests Results

Test Nb.	Cyclic Strain Amplitude - γ_{cyc} (%)	Dissipated Energy-$W^{0.5}$	Number of Cycles for liquefaction -N_{liq}	Dissipated Volume (ml)
Test 1	0.253	0.83	20	4.23
Test 2	0.3	0.72	10	4.86
Test 3	0.39	0.72	7.5	4.57
Test 4	0.175	1.55	70	3.40

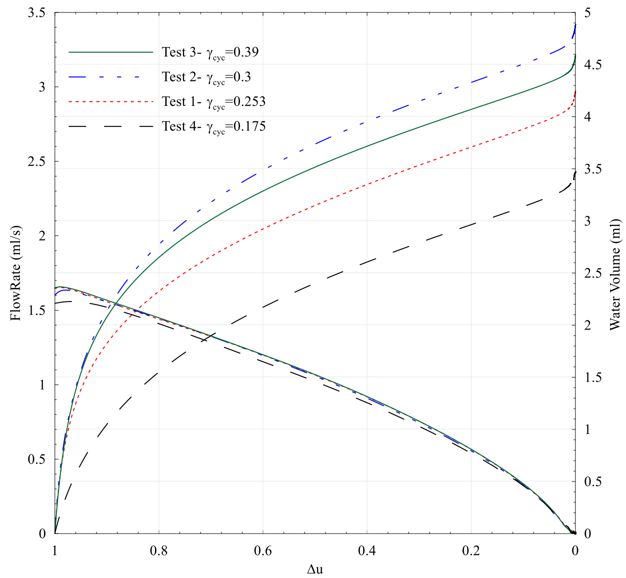

Figure 5. Variation of flowrate and volume as a function of Δu

4.2 Post-Cyclic Hydraulic Conductivity Variation

The post liquefaction phase is experimentally characterized by the pore pressure dissipation after the cyclic loading. The loss of soil particles' contact during the cyclic loading yields to their rearrangement and thus changing somehow the permeability of the tested soil specimen. The failure behavior and loading conditions as well as the soil intrinsic parameters can control the variation percentage in such permeability. However, the dissipation process acts as a reconsolidation phase, in such a way it allows the volume to change and sand particles to settle reaching the initial soil permeability or slightly different but with same order of magnitude (Bayoumi et al. 2017). From this point, the excess pore

Table 3. Measured and theoretical Hydraulic conductivity results in cm/s

Test Nb.	K initial (Pre-Cyclic) ≅ K Post-Cyclic (Measured)			K Post-Cyclic (Theoretical)
	2kPa	5 kPa	10 kPa	N/A
Test 1	0.000684	0.000411	0.000301	0.000338
Test 2	0.000629	0.000364	0.00027	0.000426
Test 3	0.000677	0.000392	0.000305	0.000402
Test 4	0.000716	0.000414	0.000314	0.000326

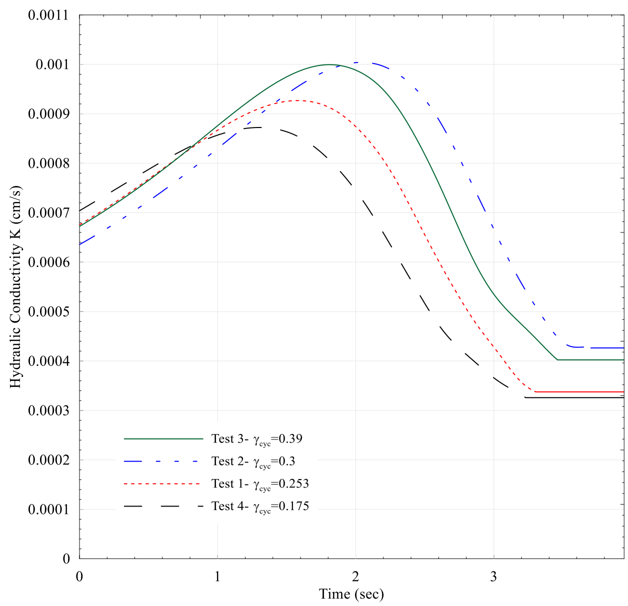

Figure 6. Hydraulic conductivity variation as a function of the excess pore pressure dissipation time history

pressure dissipation after cyclic loading seems to resemble a falling head test. The availability of volume, flowrate and pore pressure measurements results in theoretically calculating the hydraulic conductivity of the soil specimen during dissipation as shown in Figure 6. It can be noticed in Figure 6 that at the beginning of the test the hydraulic conductivity seems not to be increased significantly during the cyclic loading, this can be explained by specimen scaling effects or the fact that the sample is allowed to collapse in a way that the particles are rearranged and redistributed in a constant volume. It is important to mention that this feature is under investigation processes and can be well analyzed by using several types of soils with different parameters and measurement trials of estimating the cyclic and after cyclic hydraulic conductivity. Beyond this value, the hydraulic conductivity starts to increase reaching approximately 1.3 to 1.5 times this value. This is explained due to space occupied by the free water adhered by the soil due to the rearrangement of the soil particles in the cyclic loading. The dissipation of the 4 tests is approximately the same, it didn't last more than 4 seconds. It can be shown from Figure 6 that the decrease of hydraulic conductivity with the dissipation time history seems to be following the same trend as the pore pressure dissipation which was affected by the cyclic amplitude. If the obtained results are all normalized to the initial value of hydraulic conductivity, the fastest rate of a hydraulic conductivity decrease is accompanied by the lower cyclic strain amplitude test which is test 4. The small rearrangement of the soil particles during the small cyclic loading of this test can explain this quick decrease in hydraulic conductivity. The measured Hydraulic conductivities (K) at the end of the dissipation process for the 4 tests have the same order magnitude and approximately similar values as those obtained from the curves. In other words, as shown in Table 3, the measured hydraulic conductivity by a constant head that mainly rely between 2 and 5 kPa is compatible with that of the theoretical one calculated from the dissipation. It is important to note that at the end of the dissipation, due to the small value of the pressure reached at the end of the test or which is referred to a Δu less than 0.05, fluctuations appears. As a result in Figure 6, the theoretical hydraulic conductivity is illustrated to be constant by solid lines for the 4 tests. On the other hand, in order to verify whether the experimentally measured K using the TxSS cell is applicable and accurate, the same relative density of Ottawa sand under the same confining pressure is tested in the conventional permeameter. It is important to note that the hydraulic conductivity results showed exactly the same order of magnitude as the K initial results shown in table 3. The Hydraulic conductivity values of Ottawa sand measured by TxSS or conventional permeameter match as well those stated in literature (Thevanayagan 2000). Where Thevanayagan mentioned that the K values range between 0.0006 cm/s and 0.0013 cm/s.

5 CONCLUSIONS AND RECOMMENDATIONS

Several studies took into consideration the hydraulic conductivity variation during the generation and dissipation of pore pressure in any numerical liquefaction analysis. However, little studies tackled this representative change experimentally. This paper elaborates the measurement of hydraulic conductivity in the post seismic activity on several Ottawa soil specimens under the same relative density using the Triaxial Simple Shear Test (TxSS) apparatus; which has been designed and constructed in the course of a collaboration project between Hydro-Québec and Université de Sherbrooke. In this paper, the hydraulic conductivity measurement accuracy concerns have been solved especially by considering the flowrate of 3 to 4 ml, and by manufacturing porous stones. The repeatability and compatibility of the results are shown for the 4 tests during the initial and final measurement of K as well as through the compatibility of the TxSS and conventional permeameter. The use of the falling head method for calculating the hydraulic conductivity variation throughout dissipation time history gives a promising result especially with the measured K results at the end of the dissipation process. However, this needs intensive investigation using several soil types and several relative densities to confirm its efficiency. This paper doesn't only question the methods of hydraulic conductivity post-cyclic analysis, but it also opens the doors for the Diameter to height ratio restrictions. Through a small overview, it is proved that the effect of the specimen size on cyclic loading quietly differs using TxSS.

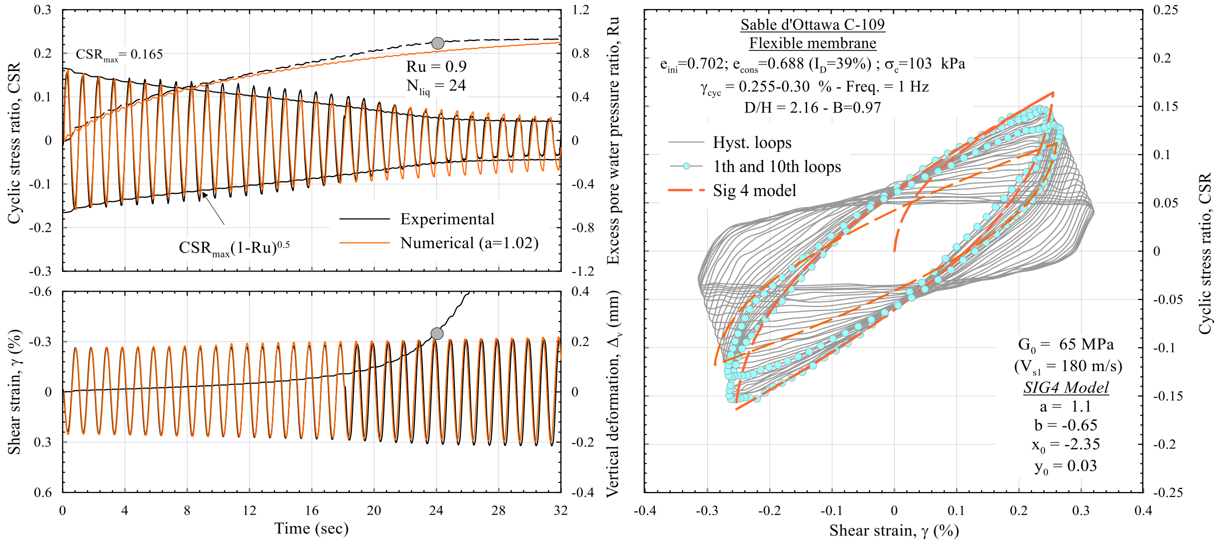

Figure 7. TxSS Records of CSR variation and pore pressure generation of Ottawa C-109

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