Investigation of the seismic effect in Eastern and Western regions of North America based on energy-based model

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ABSTRACT

Liquefaction is one of the most damaging phenomena of both infra and superstructures. Liquefaction case histories and seismic data, in general, are well documented in some regions with high seismicity (e.g., United States (Western regions), Japan, and China). However, owing to the lack of historical seismic events, hazardous data in other regions are still poorly documented (e.g. Eastern North America). In these regions (Eastern North America), liquefaction charts are obtained from Western regions and used in the geotechnical engineering design ignoring the differences in geological conditions and the earthquake characteristics (frequency and duration) between Eastern and Western regions. In fact, special attention should be paid to verify the applicability of the liquefaction charts in the eastern region and this is what will be highlighted herein. To this end, a series of experimental work has been performed using a new combined triaxial simple shear apparatus "T_xSS" on reconstituted samples of Ottawa sand F65. An energy-based pore pressure model was proposed and incorporated in the FLAC^{2D} platform. The adopted energy-based pore pressure model was validated by comparing its results to the well-established FINN model' results and a centrifuge model' results introduced by [1]. The proposed numerical model is used to compare the seismic response of soil deposits subjected to compatible and incompatible earthquake motions with the Eastern seismicity. The comparison infers the doubt of using the current liquefaction charts in Eastern regions.

Keywords: Earthquake, Liquefaction, T_xSS, Energy concept, pore water pressure ratio.

INTRODUCTION

Seismically induced pore water pressure generation in saturated cohesionless soil leads to decaying of soil stiffness and a loss of strength and eventually onset of liquefaction. This phenomenon results in devastating damage in civil infra- and superstructures such as slope, dams, and buildings. Aftermath of Anchorage, Alaska (M_w 9.2) and Niigata, Japan (Ms 7.5) earthquakes in 1964, great attention of geotechnical engineers have been devoted to understanding more deeply the liquefaction phenomena and evaluating liquefaction potential. That leads the researchers to perform in-situ and experimental investigations (e.g. [2]; [3]). The most widely used procedure among geotechnical engineers to evaluate the liquefaction potential was initially developed by Professor Seed and his colleague in California defined as "simplified method" based on a compilation of tectonic earthquake data from highly seismicity regions such as Western North America, Japan, and China ([4]). Afterwards, extensive work has been done to acquire cyclic resistance ratio (CRR) experimentally and in-situ (i.e., SPT, CPT, and Vs). The simplified procedure was incorporated in conjunction with in-situ test data to introduce seismic and liquefaction charts (e.g. [5], [6], and [7]). However, these charts are based on data collected from high seismicity regions (e.g., United States (Western regions)) where ground motion is characterized by low-frequency content (rich in long-period motions) and peak ground acceleration (PGS) comparing to their counterparts in Eastern North America at similar magnitude and source to site distance ([8]; [9]; [10];[11]). This divergence between eastern and western ground motions has a significant impact on the equivalent effective stress or strain cycles and the magnitude scaling factor (MSF), a condition that questions the applicability of the current liquefaction charts in Eastern North America. Thus, a comparison between seismic responses of a soil deposit under the application of earthquake motions compatible and incompatible with the Eastern seismicity would be a significant step towards a comprehensive understanding of the issue. This will be numerically addressed in the current study adopting an energy-based pore pressure model through the computer code, FLAC. Preliminary, the proposed energy-based pore pressure model is calibrated by means of a series of cyclic strain controlled tests performed using the new combined triaxial simple shear test "TxSS" ([12]). The numerical model is validated by comparing its results with FINN liquefaction model' results and centrifuge model tests introduced by [1] as will be discussed in the following sections.

EXPERIMENTAL WORK

During the last decades, special attention has been taken to mimic the cyclic response experimentally (e.g. triaxial, torsional shear, and direct simple shear (DSS)). Direct simple shear test (DSS) considered as the most practical apparatus in liquefaction investigation as it replicates the stress condition during vertically propagating shear waves. The first versions of DSS were developed by ([13], [14]). After that, different versions have been evolved in an attempt to overcome the shortening of typical DSS. Recently, a new combined triaxial simple shear apparatus (T_xSS) was developed by the Institute de recherché d'Hydro-Québec (IREQ) and the Université de Sherbrooke ([12]). Monotonic or cyclic (sinusoidal or irregular ground motion) can be applied on hydrostatically confined saturated soil samples under drain or undrained conditions. The generated pore pressure under undrained condition (monotonic or cyclic) can be measured directly comparing to DSS wherein the pore pressure is typically assumed equal to the change in the effective vertical stress to preserve a constant volume during cyclic loading.

In the first stage of the current study, a series of uniform cyclic strain-controlled tests were performed on reconstituted soil samples of Ottawa F65 (rounded grains) sand prepared with a diameter of 79 mm and a 26 mm height. The Ottawa sand has a specific gravity of 2.65 and a mean effective diameter, D_{50} of 0.4. The maximum and minimum void ratios are 0.81 and 0.53, respectively. The wet tamped preparation method was used to prepare reconstituted soil specimens at a relative density of 35, 40% and 90%. Each sample is prepared in three layers and each layer was gently compacted to the desired density. After saturation, with a Skempton's parameter (B) value greater than 0.96, the sample was isotropically consolidated. Then, a cyclic shear strain wave was applied with a frequency of 1 Hz on the sample's surface under undrained condition till triggering of liquefaction. The liquefaction criterion is defined in the current study as the generated pore pressure ratio ($Ru = \Delta u/\Delta\sigma_c$, Δu = excess pore water pressure) attains 0.9 ([15]). The induced shear load, pore water pressure, and vertical displacement are recorded using a high accuracy electric piezometers and displacement transducers.

ENERGY BASED PORE PRESSURE MODEL

Estimation of earthquake-induced pore water pressure is the main challenge in the numerical analysis. Variant pore pressure model were previously introduced based on cyclic strain- or stress-controlled tests as a function of loading cycles (e.g. [16]; [15]) or damage parameter (e.g. [17][18]). Following the energy concept proposed by [19] to link the soil densification in a drained test to the dissipated energy during cyclic loading, variant of pore pressure models have been developed and incorporated in the dynamic analyses(e.g. [20]; [21]; [22]). These models correlate the generated pore water pressure ratio (R_u) during cyclic loading with the dissipated energy per unit volume of soil (Ws). Where Ws is defined by the accumulated area bounded by hysteresis loops and can be yielded under simple shear tests as follow:

$$W_s = \frac{1}{2\sigma'_{mo}} \sum_{i=1}^{n-1} (\tau_{i+1} + \tau_i)(\gamma_{i+1} - \gamma_i) \tag{1}$$

In the current study, the results of strain-controlled test were adopted to develop an energy-based pore pressure model, Eq. 2, which was coded in $FLAC^{2D}$ platform to investigate the seismic and cyclic response of a level sand deposit. Fig. 1 shows a unique nonlinear relation between R_u and the Ws normalized by a calibration parameter (a) for the loose Ottawa sand employed from different shear strain amplitudes. The calibration parameter (a) is a function of shear strain amplitude ($\gamma_{cyc.}$) and can be fitted as Eq. 3. It is noteworthy that the adoption of the energy-based concept alleviates the conversion requirement of earthquake motion to equivalent uniform cycles and can be applied for both uniform cyclic test and irregular ground motion.

$$R_u = 0.11453\left(\frac{W_s}{a}\right) + 1.6739\left(\frac{W_s}{a}\right)^2 - 0.867\left(\frac{W_s}{a}\right)^3 - 0.0367\left(\frac{W_s}{a}\right)^4 \tag{2}$$

$$a = 0.5437\left(\gamma_{cyc}\right)^{-0.4} \tag{3}$$

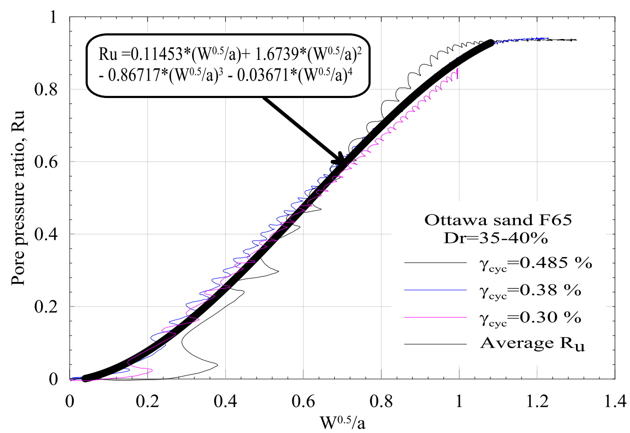

Figure 1: Ru-Ws relation after strain-controlled tests

NUMERICAL MODEL

To investigate the seismic response of the soil deposit, the proposed energy model in the preceding section is incorporated in FLAC numerical platform as a soil constitutive model. At first, a comparison between T_xSS response to the numerical simulation of strain-controlled test ($\gamma_{cyc.}$) under T_xSS condition is shown in Fig. 2 in term of generated R_u and cyclic stress ratio (CSR $=\tau/\sigma_c$). It is important to note that the shear modulus is adjusted each time step as a function of R_u, Eq. 4 ([23][24].

$$G = G_{max.}\left(1 - R_u\right)^{0.5} \tag{4}$$

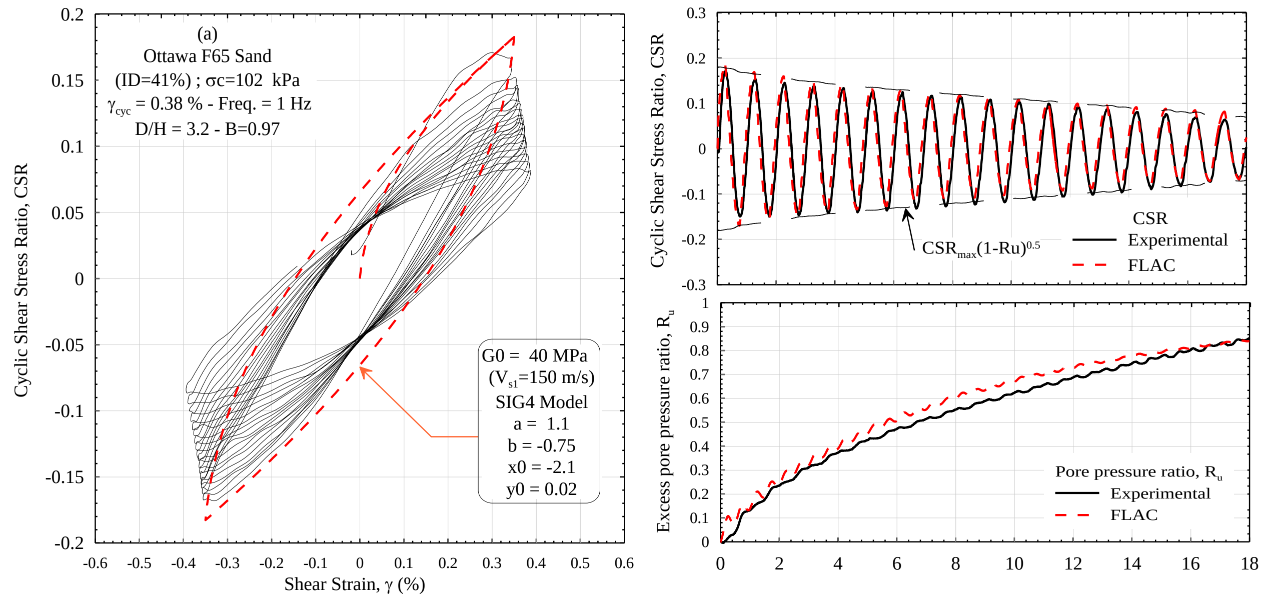

Figure 2. Typical results of TxSS test and numerical model in terms of (a) Hysteresis loops (CSR-γ_{cyc}), (b) CSR time history, (c) R_u time history.

This comparison indicates that there is an excellent agreement between the measured and the estimated R_u as well as CSR that represents a reasonable capability of the proposed model to capture the cyclic behavior of Ottawa sand F65 under T_xSS condition.

At the second phase of validation of the proposed model, before using it in the comparison between eastern and western seismicity, a centrifuge model performed at the University of Colorado Boulder (CU Boulder) by Ramirez et al. [1] is numerically simulated in FLAC^{2D} based on the prior energy-based pore pressure model. As schematically illustrated in Fig.3, the soil was placed in a flexible shear beam container using dry pluviation method. The model consists of a 12m dense layer (D_r=90%) of Ottawa sand overlaid by a 4m looser layer (40%) covered by a 2m layer of dense Monterey sand (D_r=90%). The scaled horizontal component of the 1995 Kobe earthquake recorded at the Takatori station is referred as Kobe-L was applied at the centrifuge base and the numerical model (Fig. 3b). The maximum acceleration after scaling is 0.3g. Numerically, uniform soil deposit modeled as a 1D column discretized into axisymmetric quadrilateral 2D zones and each zone thickness equal 0.5 which match the criterion of smaller than one-tenth (1/10) of the wavelength. This model was run twice, once by adopting the energy concept and the other one by using the incorporated Finn model. The comparison between the measured and estimated R_u at the top, middle and bottom of the loose layer (40%) are plotted in Fig.4. It is observed there is a good agreement between the induced and the measured pore pressure at the middle and the bottom of the loose layer. However, at the top of the liquefied layer, the pore pressure from the energy model is reasonably good till a R_u of 50% then it increases abruptly. This disturbance may be attributed to the change of soil at this transitional zone where the behavior of Monterrey sand is out of the scope of this study. It can be observed from Fig. 4 that the R_u generates after 4 seconds of the earthquake excitation. This can be attributed to that the induced cyclic shear strain value, before 4 seconds, is lower than the threshold strain value (0.01 %) wherein it is well established that the pore pressure generates only when cyclic shear strain higher than

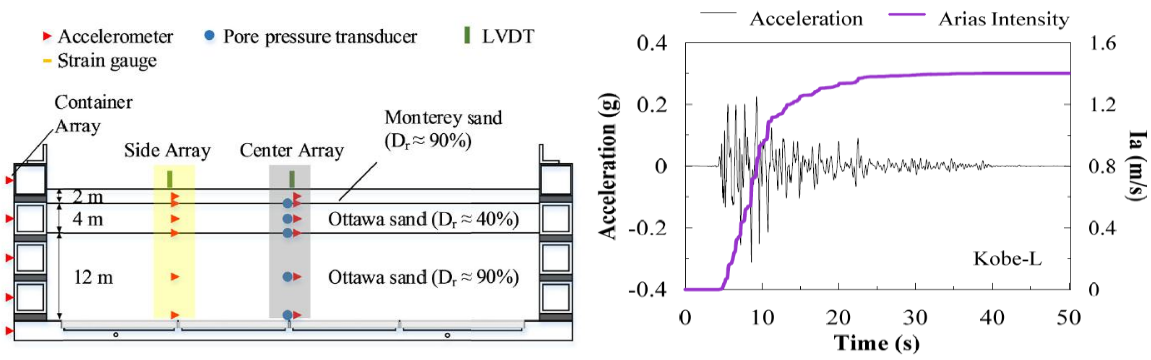

Figure 3. Schematic of the Centrifuge model and the scaled horizontal acceleration time history of Kobe-L [1].

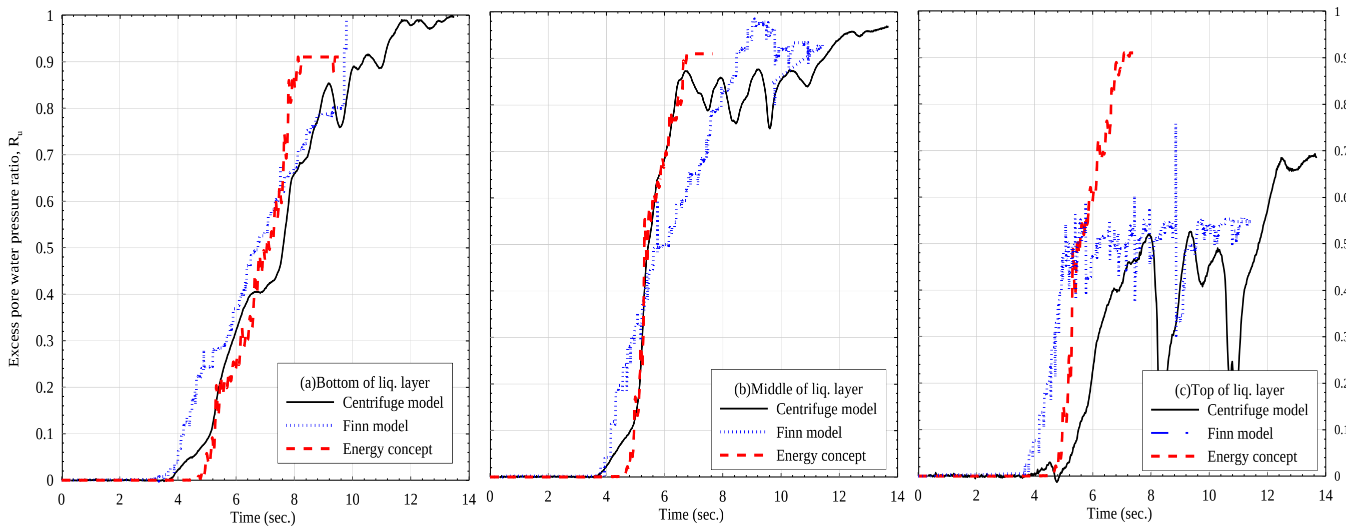

Figure 4. Comparison between the measured and estimated pore water pressure at (a) bottom (b) middle, and (C) top of the loose Ottawa sand layer.

the threshold strain value (e.g. [25]; [26]). The success in predicting the R_u in the centrifuge and T_xSS tests confirm the predictive capability of the adopted energy-based pore pressure model.

RESPONSE OF UNIFORM SAND DEPOSIT

Three hypothetical uniform loose deposits 10, 20 and 40 m thickness (D10, D20, and D40) were used herein to study the difference in the dynamic responses of soils to two different earthquakes (compatible and incompatible with Eastern Canada seismicity). The energy-based pore pressure model has been adopted in FLAC^{2D} platform to predict the dynamic response and pore pressure generation in the soil model. Soil deposits were divided into sub-layers of 0.5 thick each with a relative density of 35%. The shear wave velocity, V_s, profile is shown in Fig. 6a wherein maximum shear modulus G_{max} of each sub-layer is calculated from elastic equation $G_{max}=\rho.V_s^2$.

Two horizontal ground motions are included in the current dynamic analysis. One synthetic earthquake (Atkinson-1 [27]) compatible with seismicity of Eastern Canada and another real earthquake recorded after the Northridge earthquake, Figs 5a-b, respectively. The acceleration spectrums were scaled to fit the design spectrum class A for Eastern Quebec City region (NBCC 2005), Fig 5c. These motions were given at the base of the soil deposits to conduct 1D ground response analyses. Maximum cyclic stress ratio, CSR_{max}, and generated pore pressure, $R_{u\,max}$, along with the deposit depth after earthquake

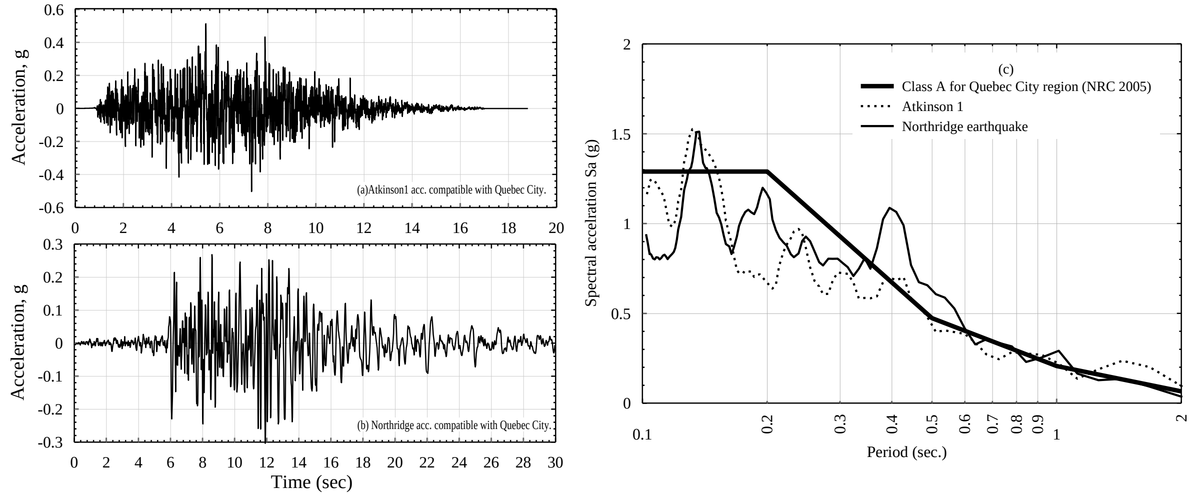

Figure 5. (a) scaled synthetic accelerograms (b) scaled Northridge earthquake to the seismicity of Eastern Quebec City. (c) Acceleration spectra for synthetic and Northridge earthquakes spectrums compatible with Eastern Quebec seismic.

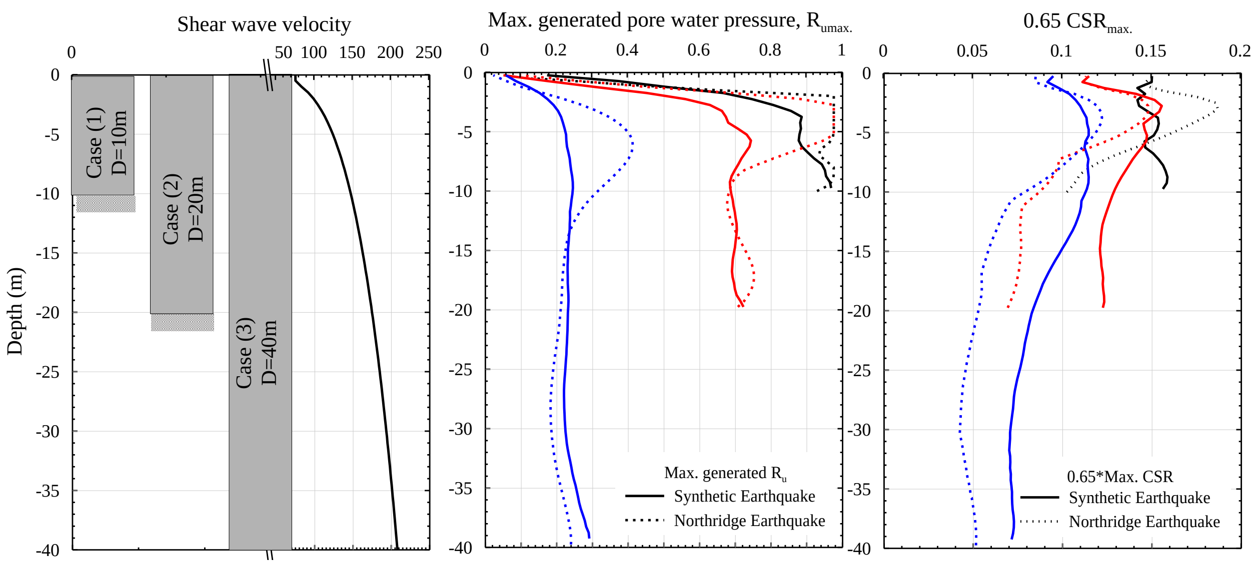

Figure 6. (a) Shear wave velocity and (b) maximum average cyclic stress ratio and generated pore water pressure ratio, R_u calculated by 1D linear analysis along depth in the uniform soil deposit by 2 earthquake motions.

excitations are shown in Fig. 6. As can be seen from Fig. 6, at different soil deposits, the greater the depth of the soil layer, the lower the CSR_{max} and the $R_{u\ max}$ are in agreement with previously reported results by [24].

Furthermore, marked differences can be inferred from the dynamic responses based on the deposit height (natural period) and earthquake characteristics. For D10, it is observed that the liquefaction occurs (i.e. Ru > 0.90) at all depth for both earthquakes. For D40, no liquefaction occurs and the $R_{u\ max}$ is 0.40 and 0.22 under Northridge and synthetic earthquake, respectively. Although the maximum CSRs are almost the same under both the earthquakes, the induced $R_{u\ max}$ values are different. These marked observations confirm that the generation of pore water pressure is affected more fundamentally by the cyclic strain amplitude than the cyclic stress amplitude [26]. However, at D20, The onset of liquefaction, $R_{u\ max}$>0.90, was observed at a depth ranged from 2 to 7 m under Northridge earthquake, however under synthetic earthquake no liquefaction occurred, $R_{u\ max}$ is about 0.7.

Figures 7a-b exemplified the computed response in terms of CSR, R_u and induced shear strain, γ, at depth 5-6 m of D20 obtained from compatible and incompatible earthquakes, respectively. This depth was selected as where the CSR_{max} has occurred, Fig 6c. It can be observed that although the CSR_{max} values are almost equal, CSR_{max}= 0.22, under two applied earthquakes, the induced $R_{u\ max}$ is different. Careful observation of Figs. 7 a-b indicates that the response of the Northridge earthquake is rich in cycles having high CSR amplitude values relative to CSR_{max} which significantly incorporated in the number of equivalent uniform cycles, N_{eq} [16]. However, the response of compatible earthquake consists of a lower number having relatively lower CSR amplitude values. In other words, at the same earthquake magnitude value, the dissipated energy, represented in the area bounded by the hysteresis loops, during earthquake loading in Eastern regions of North America is relatively low compared to its counterparts in other high seismicity regions. This reveals that the applicability of the simplified stress-based method developed Prof. Seed and his colleague, where the key parameter is CSR_{max} or maximum ground acceleration, PGA; to evaluate the liquefaction potential in the Eastern regions of North America is questionable.

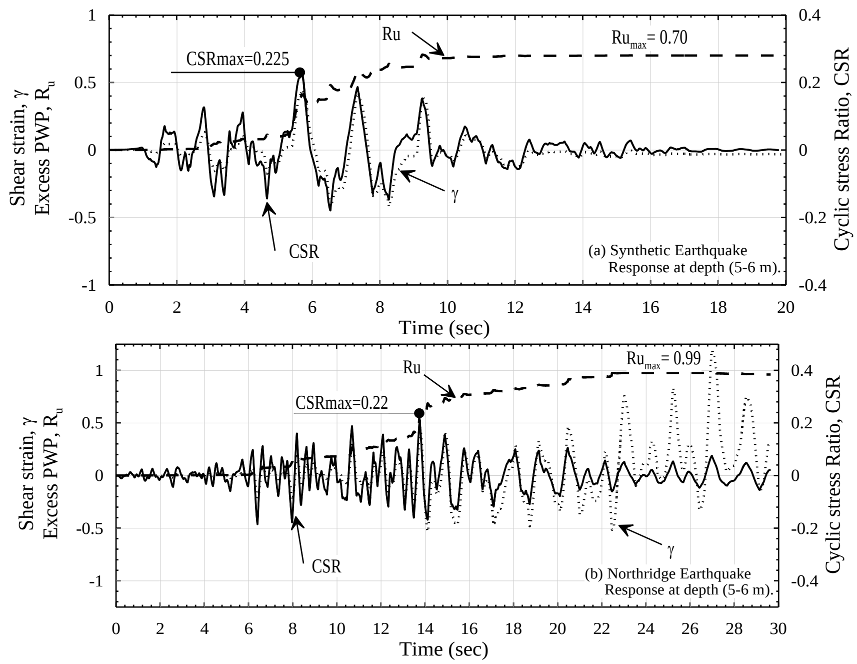

Figure 7. The cyclic response of 20.00 m deposit at 5-6 m.

CONCLUSIONS

A simple correlation between experimental and dynamic ground response can be obtained through the application of the well-known energy-concept. This correlation can be directly applied without the restriction of converting the irregular ground motion to equivalent uniform loading cycles. A new coupled energy-based pore pressure model was introduced based on a series of strain-controlled tests performed on the new combined triaxial simple shear apparatus "T_xSS". The proposed model was successfully validated to capture the cyclic response of a single element strain-controlled test and the generated pore pressure in the centrifuge model. The proposed energy-based model was adopted in $FLAC^{2D}$ platform to investigate the divergence of the dynamic ground response of different deposit heights excited by compatible and incompatible earthquakes with Eastern seismicity. Comparing the ground response reveals that the generated pore pressure is different even when CSR_{max} is equal. Although the applied earthquake records have the same magnitude, the soil may be liquefied under incompatible earthquake while no liquefaction observed due to the compatible earthquake. This infers the doubt of using the current liquefaction charts, based on the induced CSR_{max}, in the Eastern region.

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