ESTIMATING EARTHQUAKE-INDUCED PORE PRESSURE IN URAYASU CITY DURING THE 2011 EAST JAPAN EARTHQUAKE

Ziad KTEICH¹, Pierre LABBE², Jean-François SEMBLAT³, Emmanuel JAVELAUD⁴, Abdelkrim BENNABI⁵

ABSTRACT

This paper proposes two approaches devoted to quantifying pore pressure build-up and to estimating the earthquake induced settlements within a 1D horizontally layered sand deposit. In the first coupled method, we combine a pore pressure model into a conventional linear equivalent computation by taking into consideration behaviorist non-linear response due to shear strain and pore pressure increase. A further simplified version of the latter method is developed allowing us to calculate the maximum dynamic response and the maximum value of pore pressure build-up that can be reached during the earthquake, using the basic entities as mechanical characteristics and spectrum response, without running a time consuming computation. This predictive version provides fast estimation of pore pressure increase and settlements and the coupled method provides a detailed response of the profile in terms of pore pressure evolution and accumulation of settlements. These two approaches are applied on the case of Urayasu city, hit by the 2011 Great East Japan earthquake. Results are compared to actual records of settlements and liquefaction observations at different spots of the city. Numerical results are in a good agreement with the observations. The two approaches estimate reasonably the liquefaction or the no liquefaction as observed in situ.

Keywords: Liquefaction; Earthquake; coupled approach; Urayasu

1. INTRODUCTION

A great number of earth dams and sites have been severely damaged during previous earthquakes in Japan including the 2011 Great East Japan earthquake with magnitude of 9.0. One of the major phenomenon that occurred and is relatively the cause of damage, is the liquefaction of soil under cyclic loading. A new intermediate simplified coupled method that studies the dynamic response of a 1D model of a soil profile under a seismic loading by taking into consideration behaviorist non-linear response is proposed, combining a conventional linear equivalent analysis and a constitutive model developed by Byrne (1991) to quantify the risk of liquefaction and ground subsidence.

A case study of a real case, the city of Urayasu in the bay of Tokyo hit by the Great East Japan Earthquake (Mw 9.0) that occurred on 11 March 2011 and induced severe soil liquefaction in reclaimed lands of the city leading to ground subsidence, is presented. We will discuss the main features of different soil profiles and the results of our method applied on both the non-liquefied areas and the judged to be liquefied areas, respectively in terms of pore pressure build-up and ground subsidence. The results are consistent with actual observations of settlements and liquefaction risk and constitute a first validation of the proposed analysis. The development of further simplified version of

¹PhD student, IRC, ESTP Paris, Cachan, France, zkteich@estp-paris.eu
²Professor, IRC, ESTP Paris, Cachan, France, pierre.labbe@estp.fr
³Professor, Mechanical engineering, ENSTA, Palaiseau, France, jean-francois.semblat@ensta-paristech.fr
⁴Engineer, TEGG, EDF, Aix-en-Provence, France, emmanuel.javelaud@edf.fr
⁵Professor, Geotechnical department, ESTP Paris, Cachan, France, abennabi@estp-paris.eu
this analysis in which the pore pressure build-up and the earthquake-induced settlements can be predicted from given basic entities such as main mechanical properties of the soil profile, relative density and usual characteristics of the input signal as strong phase duration and response spectrum of the motion, is hereafter proposed. One of the main hypotheses accounted herein is associating the input motion to a sample of a Gaussian large-band random process and regarding the dynamic response as a sample of a narrow-band random process. This latter concept allows us to obtain an information about the number of cycles and the distribution of distortion’s values by cycles. The key concept of this predictive version is the compaction/liquefaction model proposed by Byrne (1991) used to evaluate the volumetric deformations generated by the seismic motion. The two parameters of this latter model can be inferred from the relative density and the type of soil in question. Besides, we can derive the pore pressure build-up along the profile and the global profile settlement by integration. The main advantage of these predictors, is that they provide fast estimations of soil damages in a site (settlements and risk of liquefaction), without running expensive time-history analyses. The results are compared to those of the combined method developed in this paper and validated against actual records of settlements at different spots in the city of Urayasu.

2. CITY OF URAYASU

The city of Urayasu consists of three parts, Moto-machi, Naka-Machi and Shin-Machi, classified in terms of reclamation age: the oldest part Moto-Machi is a natural land, the two other lands are in turns reclaimed areas after 1964 and until 1980. Authors as Konagai et al. (2012) and Tokimatsu et al. (2012) and Ishihara et al. (2011) conducted several surveys to evaluate damages and to summarize the liquefaction and non-liquefaction areas on representative maps. Based on these observations, no liquefaction was detected in the old town of the city including the neighborhood of K-NET Urayasu station. Sand boiling, ground subsidence, sinking of wooden houses were extensively observed in the reclaimed parts.

2.1 Geotechnical data

Chiba Geoenvironmental Information Bank provide a wide data of many boreholes located in the different areas of the city of Urayasu. Our study includes twelve soil profiles selected in a way to cover the different parts of the city (old city and reclaimed area), liquefied and non-liquefied areas and spots where there are real observations provided by Ishihara et al. (2011). Two representative models are produced and will be presented in the current paper. Location 1 – where there was no liquefaction and location 2- where there was liquefaction. Location 1- was based on a borehole in the Fujimi 3 area (investigation No. 22344 Chiba GeoInf. Bank), in the old town, northwest of the city. Location 2- was based on a borehole in the quarter Takasu 4-2 of the reclaimed area of the town (Investigation No. 25662 Chiba GeoInf. Bank). Figures 1 and 2 represent the stratification, Vs profile and the N-value profile respectively in location 1 and 2.

![Figure 1. N-value depth distribution, Vs profile and stratification at Location 1 based on a borehole in Fujimi 3.](image-url)
2.2 Dynamic properties

The dynamic deformation soil properties, the so-called degradation curves (G/G₀-γ) and (D-γ) were set based on Reference (Kanogai 2012): Konagai studied in his works a soil profile of a city near Urayasu, and used materials with same geotechnical and Vs profiles like we’re using in our method. Referring to his works, these degradation curves were obtained from laboratory tests on samples taken from nine depths on the site. These curves presented in Figure 3 were compared to modulus decay and damping curves proposed by the literature as Seed et al. (1986) for sands, to verify their reliability.

2.3 Seismic motion

In Urayasu city, strong ground motion was recorded at K-NET station CBH008, shown in Figure 4 (a). The observations assured that no liquefaction occurred at the K-NET Station. We conducted an equivalent linear dynamic analysis to determine the rock outcrop motion Figure 4 (b) at a depth of about 70m at KNET station. Secondly, we considered this rock outcrop motion as the input to the base rock of the two ground models studied.
3. COUPLED METHOD FOR ESTIMATING PORE PRESSURE BUILD-UP

3.1 Algorithm

This method is based on a combination of a linear equivalent dynamic analysis, a procedure for calculating the volumetric strain caused by cyclic shear proposed by Byrne and the pore pressure model developed by Martin et al. (1975) and improved by Wu (1996).

The iterative steps of the proposed method to evaluate the liquefaction potential of a sand layer at a depth \( z \) are the following:

- **Step 1.** Determination of the shear strain history \( \gamma(z,t) \) by a linear elastic calculation.
- **Step 2.** Calculation of the accumulated volumetric strain using Equation 1 (§ 3.2)
- **Step 3.** Calculation of the excess pore-water pressure using Equation 2 (§ 3.3)
- **Step 4.** Evaluation of the shear modulus and damping corresponding to \( \gamma_{\text{eff}} = \frac{2}{3} \max(\gamma(z,t)) \) on the degradation curves \( G/G_{\text{max}} = f(\gamma) \) and \( D = f(\gamma) \); the impact of \( r_{\text{eff}} = \frac{3}{3}r_{\text{max}} \) on the shear modulus: \( G_{k+1} = G_{0\text{max}}(1 - r_{\text{eff}})^n \cdot f(\gamma_{\text{eff}}) \).
- **Step 4.** Convergence Test: If \( \left| \frac{G_{k+1} - G_k}{G_k} \right| > \theta \) (\( \theta \) is taken equal to 5%) return to the step 1 with the new \( G_{k+1} \). Otherwise if convergence is reached, post processing is carried out.

The analyses were conducted using the finite element computer program Code Aster.

3.2 Computation of volumetric strain

During uniform cyclic loading \( (\pm \gamma_0) \), plastic volumetric strain is generally accumulated at each cycle or at each half cycle of cycle, so the excess pore-water pressure as well. Under seismic excitation, considered as non-uniform cyclic shearing loading, we evaluate plastic volumetric strain at points of reversal shear strain. To calculate this increment, Martin et al. (1975) suggested a four-parameter equation that was simplified into two-parameter formulation by Byrne (1991) (Equation 1):

\[
\frac{\Delta \varepsilon^p}{\gamma_0} = C_1 e^{-\frac{C_2 \varepsilon^p}{\gamma_0}}
\]

Where \( C_1 \) and \( C_2 \) are the model parameters. These two parameters depend on the relative density \( D_r \) or the \( (N_1)^{60} \) of the sand. Byrne (1991) suggested empirical relations to identify \( C_1 \) from \( (N_1)^{60} \) and \( C_2 = 0.4/C_1 \).
3.3 Pore pressure model

Martin et al. (1975) consider that volume change in drained cyclic tests are related to pore-water pressure increase during undrained cyclic loading, only in simple shear test conditions, which best simulate field deformations induced in horizontal sand deposits by earthquake excitation. Therefore, by assuming water to be effectively incompressible compared to the sand skeleton, Martin proposed the following relationship for computing pore-water pressure increase (Equation 2):

$$\Delta u = E_r \Delta e^0_v$$

(2)

Where $E_r$ is the rebound modulus of the sand skeleton. Martin et al. (1975) proposed an equation to calculate the unloading–reloading modulus derived from monotonic tests. Bhatia (1980) shows in her works, which suggested to calculate the rebound modulus under cyclic loading conditions, that the formula of $E_r$ proposed by Martin et al. (1975) overestimates the pore pressure increase.

Using the concept that the potential volumetric strain required to trigger a liquefaction only depends on the relative density of the sand and regarding that both formulations suggested respectively by Martin et al. (1975) and Bhatia (1980) need experimental data, Wu (2001) proposed an equation to calculate $E_r = K\sigma'$, where $K$ is a function of $D_r$ or $(N_1)_{60}$. The validation of this suggestion was done against experimental and in-situ data (Wu 1996, 2001).

3.4 Results

Figure 5 shows the maximum shear strain profile and excess pore pressure at the location 1. Liquefaction was not confirmed in the soils at this location (Ishihara 2011). The shear strain profile shown in figure 5, induces a maximum of 55% of excess pore pressure, a value below the level of liquefaction. From this it can be seen that the suggested method provides results with general agreement with observations.

Figure 6 shows maximum shear strain and pore pressure distribution by depth in the soil model at location 2. The liquefied layer was the reclaimed sand F with $V_s=100$ m/s. The shear strain in the liquefied layer reached 1% of deformation.

![Figure 5. Maximum shear strain distribution and excess pore pressure in Location 1.](image-url)
Calculated settlements are given in Table 1. In liquefied area (Location 2), results underestimate severely the real measurements. This gap can be justified by the fact that authors as Fukutake (2013) measured the amount of settlements in Urayasu a few days after the occurrence of the earthquake, so after liquefaction and sand boiling. However, the behavior of liquefied sand is not taken into consideration in our method and the domain of reliability of the approach ends at liquefaction. Therefore the numerical results represent pre-liquefaction settlements. Calculated settlement in non-liquefied area agrees perfectly with actual measurements. Focusing on the difference between the amount of settlements in non-liquefied areas and liquefied areas, can give us an idea about the tendency of sand to settle more when liquefied.

Table 1. Settlements calculated and measured in Locations 1 and 2

<table>
<thead>
<tr>
<th>Profile</th>
<th>Calculated Settlement</th>
<th>Measured Settlement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Location 1</td>
<td>1 cm</td>
<td>[0-5] cm</td>
</tr>
<tr>
<td>Location 2</td>
<td>5 cm</td>
<td>[20-50] cm</td>
</tr>
</tbody>
</table>

4. SIMPLIFIED VERSION OF THE METHOD

The method proposed in paragraph 3, gives satisfactory results comparing with the in-situ observations. Despite its efficiency, it is a time consuming approach and a complete analysis as it provides a transient response in the deposit. However in practice,

4.1 Hypothesis of simplification

This proposed approach is based on following hypothesis:
- The input motion is regarded as a sample of a Gaussian large-band random process and the dynamic response as a sample of a narrow-band random process;
- The maximum shear strain developed can be computed using the spectrum response at the base and the modal characteristics of the profile.
4.2 Algorithm

The idea of this version consists in relying on basic entities such as main mechanical properties of the soil profile, parameters of Byrne model adjusted by the relative density and type of the soil, and usual characteristics of the input signal as strong phase duration, response spectrum, to evaluate liquefaction potential with running neither a nonlinear computation, nor the coupled method proposed previously in section 3. This approach implements also an iterative procedure. At each iteration, the calculation steps are as follows:

- Modal analysis of the 1D-soil profile overlying a bedrock, to calculate the eigen vector $\phi_i(z)$ at a depth $z$ of the model, the participation factor $p_i$ and the eigen frequency $f_i$ of the profile.
- Calculation of the strong phase duration and the equivalent number of cycles of the seismic motion. (§ 4.2)
- Using the results of the modal analysis and the spectrum response of the motion $S_a(f, \xi)$, the maximum response is defined as following (Equation 3):

$$\gamma_{\text{max}} = \frac{dp_1}{dz} p_i \frac{S_a(f_i, \xi)}{\omega_1^2} \Gamma_{\text{max}}$$

These maxima are used to calculate the plastic volumetric strain using Byrne model.
- The shear moduli are updated taking into account the maximum deformation.
- The algorithm converges by a change of the shear moduli less than 5%.

4.3 Description of cycles

Concerning displacements and deformations, the response of the soil profile is mainly due to the contribution of the first Eigen mode (this preponderance of the first mode is less clear when one is interested for example in the response in acceleration). One consequence is that at each point at depth $z$ of the 1D model, the shear strain $\gamma(z, t)$, can be considered as a sample of Gaussian random process with narrow band. In terms of the number of equivalent deformation cycles, we consider the duration of the strong phase of the excitation signal, $T_F$, and we deduce the number of cycles by dividing this duration by the natural period of the first model mode (Equation 4):

$$N_F = \frac{T_F}{T_1}$$

In terms of amplitude, this process is at zero mean and is characterized by its standard deviation $\sigma_\gamma(z)$ (assumed to be independent of time because the amplitude of the cycles is averaged over the duration of the strong phase). The amplitudes of shear strain cycle’s $\gamma_a$ are supposed to be distributed according to a Rayleigh probability density function $q_\gamma$ (Equation 5):

$$q_{\gamma_a}(z, \gamma_a) = \frac{\gamma_a}{\sigma_\gamma(z)} \exp \left( - \frac{\gamma_a^2}{2 \sigma_\gamma(z)^2} \right)$$

Where $\gamma_a$ represents the maximum shear strain (amplitude of a cycle).

4.4 Volumetric strain for non-uniform loading

The Byrne model (Byrne 1991) is developed for constant amplitude cyclic shear loading. Contrariwise, a seismic signal is a non-uniform signal. In this case the model can be applied cycle by cycle independently. Since our method consists in estimating the final pore-water pressure increase and not treating the signal cycle by cycle, we will conduct a statistical study to draw the accumulation of plastic volumetric strain by number of cycles for different values of standard deviation $\sigma_\gamma$. 

7
4.5 Statistical study

We conduct a statistical study to re-draw the accumulation of volumetric strain for different values of standard deviation $\sigma_\gamma$.

4.5.1 Volumetric strain evolution

Figure 7 shows the expected information to get from this study. For each value of $\sigma_\gamma$, we can get the values of mean volumetric strain accumulated and the interval of variation of $\varepsilon_v$ after N cycles identified as $[\bar{\varepsilon}_v - \sigma_{\varepsilon_v}; \bar{\varepsilon}_v + \sigma_{\varepsilon_v}]$.

Therefore, we generated one hundred sequence of maximum shear strain that follow the Rayleigh distribution for 8 different values of $\sigma_\gamma = [2.10^{-4}, 3.10^{-4}, 5.10^{-4}, 7.10^{-4}, 1.10^{-3}, 2.10^{-3}, 3.10^{-3}, \text{ and } 4.10^{-3}]$. This set was chosen to cover all possible levels of shear strain ($\gamma_{\text{max}} = [5.10^{-4}, 1.10^{-3}]$) that can be induced by an earthquake and suspicious to generate a pore-water pressure increase. We applied the formula proposed by Byrne (1991) to the eight hundred samples we have, and we drew the evolution of their volumetric strain during the one hundred cycles. Samples with the same value $\sigma_\gamma$ have a similar evolution tendency, so we can reduce the 100 curves of each $\sigma_\gamma$ to their mean with a deviation value to be determined. Figure 8 present the evolution of the mean value of the volumetric strain during N cycles.

Graph illustrated in Figure 8 will be used to predict the amount of plastic volumetric strain induced by a defined signal represented as a sequence of maximum shear strain values that follow a Rayleigh distribution. In practice, we will not have exact values of $\sigma_\gamma$ as the ones existing in the statistical study,
so we tried to find a relation between curves with different values of $\sigma_i$ by normalizing $\bar{e}_v$ by $\sigma_y$ for volumetric strain mean values presented in Figure 9.

![Figure 9. Common curve of normalized $\bar{e}_v$ by $\sigma_y$.](image)

4.5.2 Peak Factors

As mentioned before, the seismic motion applied at the bedrock is generally associated to a sample of a Gaussian large-band random process, so its strong phase is considered as stationary. The soil profile responding on the first mode, filters the motion and the response in terms of shear strain $\gamma (z, t)$ is regarded as a sample of a narrow-band random process. Moreover, this response can be regarded as a set of N cycles whose magnitudes are distributed according to a Rayleigh probability density function. The standard deviation of the process and the expectation of its maximum $\gamma_{\text{max}} (M)$ are related by the peak factor ($p$) (Equation 6):

$$E (\gamma_{\text{max}} (M)) = p \sigma_{\gamma} (M)$$

(6)

For the duration of a strong seismic motion and for typical Eigen frequencies of soil deposits, the value of the peak factor $p$ is provided by the graphs of Der Kiureghian (1979).

4.6 Application on Urayasu city

We applied our predictive method on the same test case and profiles as the coupled method. Figure 10 shows the spectrum response of the outcrop signal that was used in the predictive method.

![Figure 10. Spectrum response normalized by the PGA ($\Gamma_{\text{max}}$) for 5% damping.](image)
We applied the prediction method on the two representative models selected in Urayasu. The excess pore-water pressure is only calculated for the upper layers that could liquefy and significant settlements could take place.

We calculated the volumetric strain profile in the both locations. By integrating these volumetric deformations along the problematic layer, we got induced settlements.

At location 1 (Fujimi 3), observed by Ishihara (2011) as a no-liquefaction location, excess pore pressure ratio reaches 0.38. However, excess pore pressure rate reaches 0.98 at location 2, classified as liquefied area (Ishihara 2011). The results of this simplified version of the method underestimate the results of the coupled method but they still agree reasonably with the observations.

Figure 11. Excess pore-pressure predicted at location 1.

Figure 12. Excess pore-pressure predicted at location 2.
5. CONCLUSIONS AND PERSPECTIVES

In this paper, we present two new methods to study the phenomenon of liquefaction in a 1D soil column under seismic motion. The first consists in combining a pore pressure model and a conventional linear equivalent procedure. It allows us to take in account the behaviorist nonlinear response of a 1D model caused by shear strain and excess pore pressure. The other approach is a simplified version of the first method. The coupled approach consists in running a harmonic dynamic analysis where the sand deposit is modeled and we calculated the transient response at each depth $z$ in terms of shear strain $\gamma(z,t)$. Other variables can be calculated as the time history of acceleration. Otherwise the simplified predictive version allows us to compute the maximum response at the depth $z$ by the spectral analysis without running transient computation. This latter method is efficient in engineering practice where the seismic motion is provided as a response spectrum. Both methods can predict the rate of liquefaction calculated as the increase in pore-water pressure and the induced settlement by using the basic mechanical properties of the soil, SPT results and the main characteristics of the seismic motion. The predictive version provide us a preliminary idea of the risk of liquefaction in a sand deposit and the detailed coupled approach is valorized for a precise dynamic and mechanical study. Byrne model was built based on the results of simple shear cyclic tests where a uniform shear strain with constant amplitude is imposed and volumetric strain are measured during loading cycles. However, a seismic motion induces non-uniform strain history. This history was treated as a sequence of half-cycles with constant amplitudes in the coupled approach and regarded as a sample of a narrow-band random process in the predictive version where the amplitudes of this shear strain are distributed according a Rayleigh density function. Moreover, the city of Urayasu was mainly the test-case to validate our two approaches. Generally it provides reasonable results. Nevertheless, it must be recognized that these results are validated with qualitative observations and have to be also compared to a time-history analysis.

We end this section noting that the proposed approaches providing excess pore-water pressure profile and settlements could be extrapolated to any kind of profile with different stratifications and subjected to different types of motions.

Moreover, some studies based on different type of seismic motions or on a different geometrical model as a 2D dam model are worthy of undertaking.

6. REFERENCES


