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Estimation of settlement after soil liquefaction for structures built on shallow foundations

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ABSTRACT

Soil liquefaction can occur when the strength and stiffness of soil are reduced by changes in the stress condition, and the settlement of structures can be affected by this phenomenon following an earthquake. To evaluate the liquefaction-induced settlement of structures, a simple equation that proposed the concept of equivalent viscosity of soil-water mixture is adopted. In this study, comprehensive finite element analyses are conducted and a viscosity chart is proposed that enables prediction of the liquefaction-induced settlement value. To accommodate different site conditions, several corrections are made based on experimental data obtained from literature; such corrections are found to be effective for proficiently estimating the liquefaction-induced settlement under differing conditions.

1. Introduction

The Meinung earthquake occurred in southern Taiwan in 2016 and caused considerable loss of life and structural damages. Shallow foundations are widely adopted for residential buildings in central and southern Taiwan, and many of the buildings that tilted and settled following the earthquake had been built using this foundation type. The evaluation and prediction of liquefaction-induced settlements is thus necessary to enable shallow foundations to be efficiently designed. However, the procedure used to estimate liquefaction-induced settlement is fairly complicated. Previous studies on liquefaction-induced settlement can mostly be divided into four types, numerical analyses, empirical methods, experimental studies, and simplified methods, and these are briefly presented in the following paragraph.

In early numerical analysis studies, an assumption of linear elasticity behavior was common. However, this assumption neglects the degradation of soil stiffness during liquefaction and causes an amplification of surface motion. Recently, the influences of soil non-linearity on soil-structure interactions have been investigated by many researchers (Shahir and Pak [1]; Dashti and Bray [2]; Karimi and Dashti [3,4]). However, to conduct a numerical analysis, it is necessary to have an appropriate simulation tool, an experienced operator, and a long period of computing time, which is not always practical.

Empirical methods (Ishihara and Yoshimine [5]; Tsukamoto and Ishihara [6]) for estimating liquefaction-induced settlement is based on free field results, which means that the effects of structures are excluded, even though liquefaction-induced settlement has been found to be associated with the width and weight of foundations (Yoshimi and Tokimatsu [7]; Liu and Dobry [8]; Dashti et al. [9,10]). Studies that have conducted experiments (Yoshimi and Tokimatsu [7]; Liu and Dobry [8]; Dashti et al. [9,10] and Elgamal et al. [11]) using physical modeling tests (centrifuge test and shaking table test) provide valuable information, but it is too costly to conduct such tests for every project. Furthermore, there are no current, widely accepted, simplified methods are available for estimating liquefaction-induced settlement.

Therefore, this study proposes a method of estimation based on the simple equation of Sawicki and Mierczyński [12] that was developed to estimate liquefaction-induced settlement, where the effects of soil dynamic behavior, seismic activity, and soil-structure interaction are incorporated into the equation through a viscosity parameter for soil. As it is reasonable to incorporate soil behavior and seismic characteristics in estimations, the Sawicki-Mierczyński equation is employed in this study. However, the range of viscosity of liquefied soil, the only unknown parameter in the equation, is not clearly defined and suggested in their paper. A viscosity chart is thus provided in this paper for use by engineers when assessing viscosity, as this enables a speedy evaluation

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of liquefaction-induced settlement.

2. Equation of liquefaction-induced settlement

Based on the observations from cyclic triaxial tests, Sawicki and Mierczyński [12] showed that the strain rate of liquefied soil is constant. A shaking table test was conducted as part of their research. A cylinder (25 cm in diameter) was filled with saturated sand and placed on the shaking table. A steel cylinder was placed on the ground surface. The vertical displacements of steel cylinder were recorded while the cyclic loading was applied. From the observations of this shaking table test (small-scale experiment), the sinking rate of a small rigid block is also nearly constant. Therefore, the viscosity in relation to these two kinds of test can be back calculated using the Sawicki–Mierczyński equation. The value of viscosity in cyclic triaxial tests and small-scale experiments are similar. With these results, the viscosity is considered to be an approximate value (10^6 N/m^2). Fig. 1 (a) shows that the rectangular block (structure) is under initial equilibrium prior to liquefaction; in addition, the time is assumed to be zero at this moment, and Q is the gravitational force of the rigid block. In Fig. 1(b), the block is in a condition of sinking into the liquefied soil, and the gravitational force (Q), buoyant force (W), and damping force (V) are balanced at each time step during this condition, which enables the governing differential equation to be obtained. The governing equation is listed below.

$$\mathrm{Q} = \mathrm{W} + \mathrm{V} \tag{1}$$

W is the buoyant force and is given below, where γ_m is the moist unit weight of liquefied soil; B and L are the width and length of the block, respectively; and z is the embedded depth of the block.

$$\mathrm{W} = \gamma_m \mathrm{BLz} \tag{2}$$

When an object is sinking into a viscous liquid, a viscous damping force appears. Applying this concept in the problem of heavy object in liquefied soil, Sawicki and Mierczyński (2009) proposed the form of the viscous damping force. It can be expressed as:

$$\mathrm{V} = 8\overline{D}\eta \frac{dz}{dt} = \zeta \frac{dz}{dt} \tag{3}$$

V is the resultant damping force relating to the viscosity of the liquefied soil. η is the viscosity of liquefied soil, which is a key parameter in this article. $\overline{D}$ is the substitutional diameter of rectangular base of a building. It can be expressed as:

$$\overline{\mathrm{D}} = 2\sqrt{\mathrm{BL}/\pi} \tag{4}$$

The back analyses are conducted to obtain the viscosity chart. From the viscosity chart, information is needed including soil characteristics (relative density and soil permeability), magnitudes of acceleration induced by earthquake, and environmental conditions (water level). It incorporates the effects of soil characteristics, seismic activity, and environmental conditions. dz/dt is the sinking rate of the block; and ζ is a combination of 8 $\overline{D}$ η to enable quicker calculation.

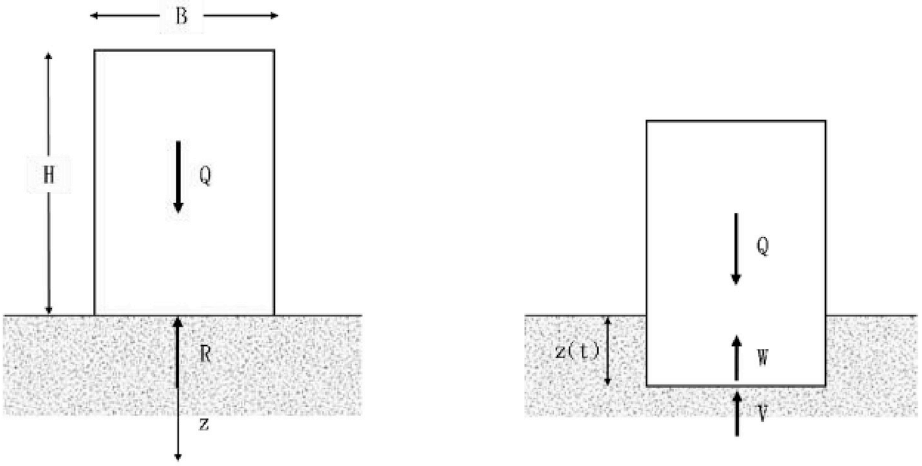
When all the parameters have been substituted into the governing differential equation, the solution (Eq. (3)) to the differential equation can be obtained as follows:

$$z\left(\mathrm{t}\right) = \frac{b}{a}\left[1 - e^{-at}\right] \tag{5}$$

where a is the combination of γ_mBL/ζ, and b is the combination of Q/ζ. Both parameter a and b contain the viscosity. The viscosity is used for evaluating liquefied settlement.

2.1. Sensitivity analysis

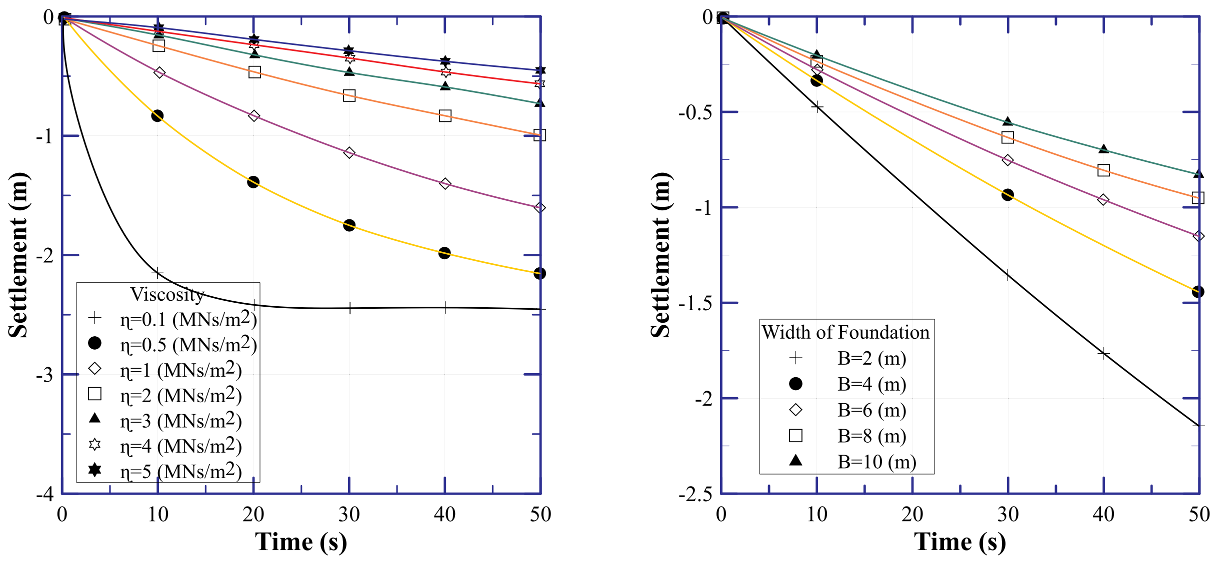
The estimation method is based on the Sawicki-Mierczyński equation and it requires six parameters: time duration, viscosity, moist unit weight of soil, foundation width, foundation length, and surcharge from structure. The sensitivity of parameters also needs to be known. In this respect, it is assumed that the foundations are not inclined. As the sensitivity of the foundation length is the same as that of the foundation width (2-D problem), only the sensitivity of the foundation width is shown herein. The time duration (t) is set to be constant in the sensitivity test (50 s). Therefore, the sensitivity of four parameters out of the six required are presented (Figs. 2 and 3). Fig. 2 shows that settlement is sensitive to viscosity and width of the structure, and settlement decreases when viscosity increases. Fig. 3 (b) shows that settlement is less sensitive to the unit weight of the structure than to viscosity. The effect of the structure's weight on liquefied-induced settlement can be included; however, it may be difficult to reflect this occurrence when using an empirical method.



(a) Before liquefaction

(b) During sinking in liquefied soil

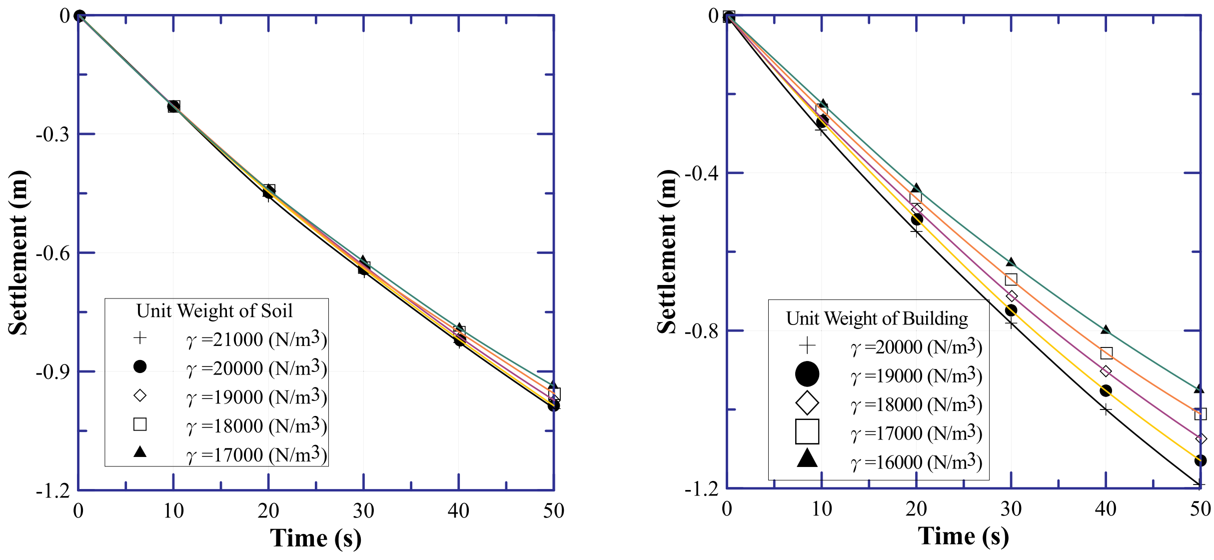
Fig. 1. Forces applied on block before liquefaction and during sinking process.



(a) Viscosity

(b) Width of foundation

Fig. 2. Sensitivity analysis of parameters.



(c) Moist unit weight of soil

(d) Unit weight of structure

Fig. 3. Sensitivity analysis of parameters.

3. Three-dimensional finite element analysis

Oka and other authors proposed a model that can produce consistent results with experimental results [13–15]. In this elastic-plastic model, both of the anisotropy of initial stress state and the initial shear stress of soil can be considered. Further, the decrease in shear modulus as increasing strain during cyclic loading was also considered in this model. Oka et al. solve liquefaction problem in the view of soil-water coupling. In finite element (FE) analysis, a soil-water coupled problem is formulated based on a u-p formulation and is employed herein. The equilibrium equation for the mixture is derived as follows:

$$\rho \ddot{u}_i^S = \sigma_{ij,j} + \rho b_i, \tag{6}$$

where ρ is total density; $\ddot{u}_i^S$ is acceleration of the solid phase; σ_{ij} is the total stress tensor; and b_i is the body force vector. The continuity equation is written as follows:

$$\rho^f \ddot{u}_{ij}^S - p_{,ii} - \frac{\gamma_w}{k}\dot{\varepsilon}_{ii}^S + \frac{n\gamma_w}{kK^f}\dot{p} = 0, \tag{7}$$

where ρ^f is the density of fluid; p is the pore water pressure; γ_w is the unit weight of the fluid; k is the coefficient of permeability; ε^S_{ii} is the volumetric strain of the solid phase; n is porosity; and K^f is the bulk modulus of the fluid phase.

The constitutive equation used for sand is a cyclic elasto-plastic model that is widely used for reproducing the cyclic undrained behavior of sand and was obtained through a comparison of numerical results and hollow cylindrical torsional shear tests. Oka et al. [13] found that the model succeeded in reproducing the experimental results under various stress conditions, such as isotropic and anisotropic consolidated conditions, with and without the initial shear stress conditions. Japanese standard sand, Toyoura sand, represents the composition of the ground in the FE analysis conducted in this paper, and for the sake of convenience, the parameters of the soil model are for Toyoura sand with a relative density of 50%, 60%, 70%, and 80%; these parameters are confirmed by the experimental data presented in the research of Oka et al. [13] and Oka et al. [14].

3.1. Numerical model setup

The constitutive model is adopted in LIQCA. LIQCA is a 3-D soil-water coupled dynamic analysis. In LIQCA, a soil-water coupled problem was based on u-p (displacement of the solid phase-pore water pressure) formulation incorporating an elastic-plastic kinematic hardening model. The finite element method was used for the spatially discretization of the equilibrium equations, and the Newmark method was used for time integration. The numerical model configuration in LIQCA is shown in Fig. 4. To obtain the viscosity in various conditions, the following were changed for each calculation case: weight of the structure loading on the shallow foundation; relative density of soil; and horizontal earthquake acceleration. The soils were modeled using 8-node isoparametric solid elements that contained 1001 nodes and 780 elements in a numerical mesh. The authors used the cyclic elasto-plastic model to represent all soil layers, and the parameters of each soil layer are shown in Table 1 and Table 2. The elements below the water table were treated as fully saturated elements with the degree of freedom (DoF) of pore water pressure.

To avoid unnecessary echo-vibration and to simulate the boundary conditions of the laminar box, the bottom of the mesh was set to be rigid and all lateral boundaries were set with equal-displacements. The input acceleration was set at the rigid bottom boundary. The applied seismic wave was a sine wave with a frequency of 1 Hz and magnitudes of 100-gal, 200-gal, 300-gal, and 400-gal. The lateral and bottom boundaries were assumed to be impermeable while the water table was permeable. A time integration step of 0.001 s was adopted to ensure numerical stability. Hysteresis damping of the constitutive model was used, and to describe the damping, particularly in the high frequency domain, it was assumed that Rayleigh damping was proportional to the initial stiffness. Furthermore, β and γ in the Newmark method were set as 0.3025 and 0.6 to ensure numerical stability. These steps were also mentioned in the technical paper [16] to ensure numerical stability.

3.2. Model verification

The model test (centrifuge test) initially conducted by Peng [17] was used to verify the performance of the numerical analysis in LIQCA (as shown in Fig. 5). The conditions in both the numerical analysis and model experiment were set to be the same: the width and length of foundation were equal at 6 m; the surcharge to the foundation was set as 10 kPa; the total thickness of the layers was 7 m; the soil layers were composed of Toyoura sand at a relative density of 50% from the ground to 6 m-depth and 90% from 6 m to 7 m (as shown in Fig. 6); and input loading was a 35-s shake at 228.9 gal under 0.8 Hz. The time histories of excess pore water pressure in simulation and experiment were shown in Fig. 7. As shown in Fig. 8, a large amount of settlement occurred after the 10th second. Although the dramatic increment in settlement cannot be demonstrated in the FEM analysis, the maximum settlement in the experiment was 24.54 cm and that of numerical analysis was 26.36 cm. As both results are very similar, it is evident that there is good agreement between the model test and numerical analysis.

ALID, a software, was proposed by Yasuda et al. [18] in 1999. This software was a pseudostatic approach to evaluate the liquefaction-induced settlement. The deviation of deformation between the deformation under the initial shear modulus of soil and the deformation under reduced shear modulus was the liquefaction-induced settlement. It was a simple way to evaluate the liquefaction-induced settlement. The simulated results (29.36 cm) overestimated the experimental result (24.54 cm) [17] that could provide a conservative approach.

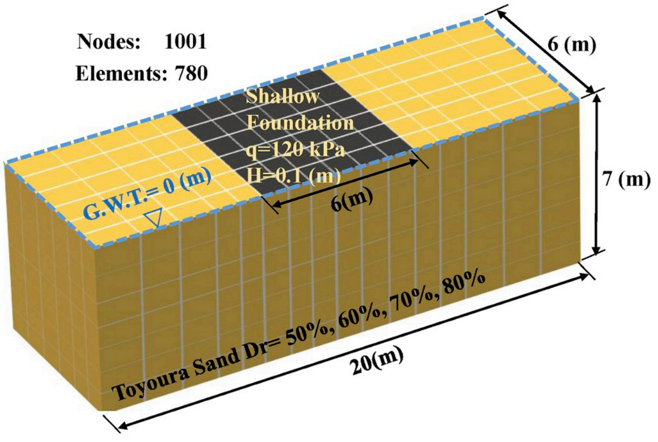


Fig. 4. Schematic diagram of FEM model in LIQCA.

4. Determination of viscosity from results of numerical analysis

Viscosity varies between locations with respect to soil density and earthquake acceleration. In this study therefore, a series of numerical analyses were conducted to obtain settlement values in different situations. Settlement and an effective stress decreasing ratio (ESDR) versus time are used in the numerical analysis. In Fig. 9 (a), σ'_m is effective stress, σ'_{m0} is total stress, and the Y axis of Fig. 9 (a) is the excess pore water pressure ratio, which is represented by ESDR in this study. When ESDR equals 1, soil reaches a liquefaction state in the general concept. However, in this study, soil is assumed to behave like a liquid when ESDR equals 0.8, which is the timing that the large settlement of soil is discovered. With this assumption, it is more conservative in estimating liquefaction-induced settlement. By substituting different values of viscosity into Eq. (4), an approach to the slope (line of time v.s. settlement) from numerical analysis was made (Fig. 9 (b)). The way, adjusting the viscosity for the next substitution after the previous substitution until reaching a desired approach is adopted to find the viscosity in each case of numerical analysis. 112 numerical analyses from ALID were also adopted to validate the performances in LIQCA. 336 numerical analyses were conducted and their back-calculated values of viscosity are plotted on Fig. 10 (a). Each case of numerical analysis can obtain a representative viscosity. 336 representative viscosities are plotted on Fig. 10 (a). Four different magnitudes of input acceleration, that is 100, 200, 300, and 400 gal, are adopted in the numerical analysis. The regression curves are made among the results under each acceleration setting (Fig. 10(b)). A viscosity chart was subsequently proposed for use in evaluating liquefaction-induced settlement.

5. Parameter correction

To evaluate the use of the estimation method, experimental (Dashiti and Bray [2]; Dashiti et al. [10]; Peng [17] and Huang [19]) from previous research were employed as standards to observe biases in the estimation method. Four corrections were subsequently proposed to reduce biases in the estimation method, and these are presented in the following sub-sections.

5.1. Amax

In this paper, Amax is defined as the peak acceleration induced by an earthquake. When the values of Amax and the relative density of the soil are available, corresponding viscosity can be obtained using the

Table 1
Soil layer parameters (Part 1).

Name of soil profile	Density	Coefficient of permeability	Void Ratio	Compression Index	Swelling index	Normalized Shear Modulus	Stress Ratio at Maximum Compression
Unit	P (t/m^3)	K (m/s)	e_0	λ	κ	G_0/σ'_{m0}	M^*_m
$D_r = 50\%$	1.879	2.2×10^{-5}	0.8	0.025	0.0003	1150	0.909
$D_r = 60\%$	1.898	2.4×10^{-5}	0.754	0.0091	0.00052	1200	0.707
$D_r = 70\%$	1.917	2.1×10^{-5}	0.716	0.0091	0.00052	1980	0.707
$D_r = 80\%$	1.938	1.9×10^{-5}	0.683	0.0091	0.00052	1980	0.707

Table 2
Soil layer parameters (Part 2).

Name of soil profile	Stress Ratio of Failure State	Harding Parameter	Control parameter of anisotropy	Parameter of Dilatancy	Reference Value of Plastic Strain	Reference Value of Elastic Strain
Unit	M^*_f	B^*_0, B^*_l, C_f	C_d	D^*_0, n	γ^{P*}_γ	γ^{E*}_γ
$D_r = 50\%$	1.229	2000,400,0	2000	1, 4	0.005	0.003
$D_r = 60\%$	0.99	4089,54.5,0	2000	0.6, 5.1	0.002	0.012
$D_r = 70\%$	1.18	4001,100,950	2000	0.8, 7	0.0032	0.003
$D_r = 80\%$	0.99	4500,65.4,0	2000	0.52,8.5	0.005	0.025

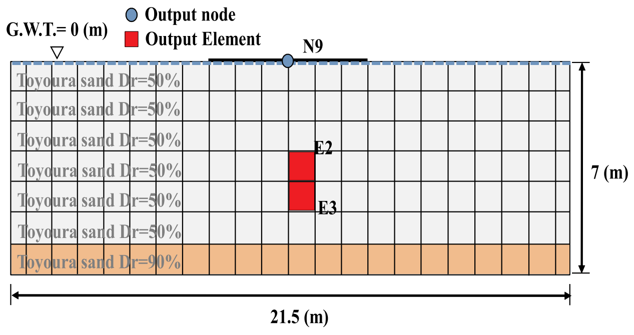


Fig. 5. Schematic diagram of LIQCA.

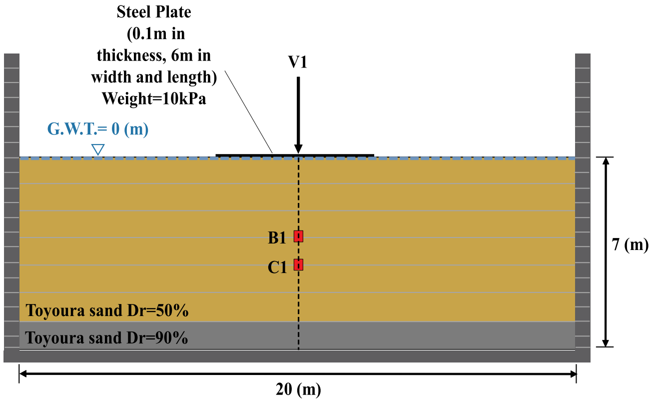


Fig. 6. Schematic diagram of model test.

viscosity chart (Fig. 10 (b)). This corresponding value of viscosity (η) from the viscosity chart is required to estimate liquefaction-induced settlement (as seen in Eq. (3)). The field data of settlements were used to compare with the estimated value by using x-y plots. It means that estimated value is perfectly consistent with the experimental data when cases located on the 100% line. However, more cases were located in the 50%–200% zone by using 0.65 Amax than Amax. With these results, 0.65 Amax as the value to obtain viscosity was found to have a better prediction than using Amax. therefore, a correction factor of 0.65 for Amax was subsequently employed.

5.2. *Time duration*

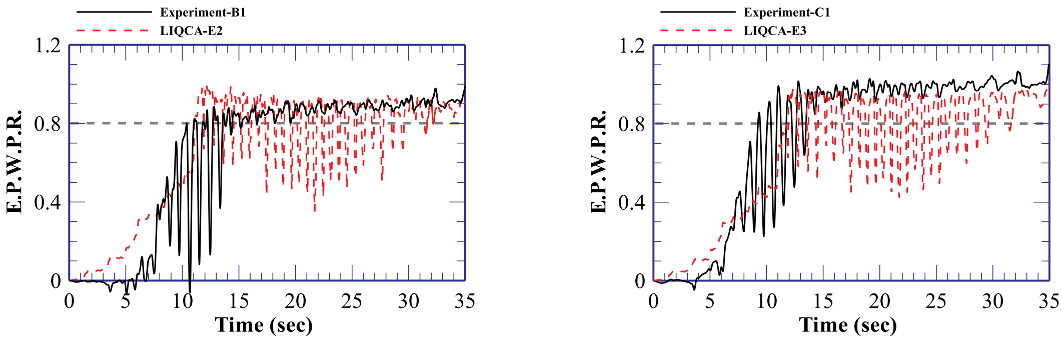
In Eq. (3), the time duration (t) is a key factor in the estimation method, and a larger settlement occurs with an increase in the time duration. In the field, it is very difficult to determine the exact timing of liquefaction during an earthquake. In our initial calculations, the total time length of an earthquake served as the time duration (symbolized as T_0 in this paper), but we subsequently found an effective correction for the time duration that improves the performance of the estimation method. Fig. 11 shows a schematic diagram of acceleration versus time using experimental data. The corrected time duration (symbolized as T_{90} in this paper) is based on the research of Trifunac and Brady [20]. T_{90} is defined as the time duration from 0.5 to 0.95 its total accumulating absolute value of a strong motion acceleration.

5.3. *Ground water level*

The water table does not always meet the ground's surface; this needs to be considered when conducting the numerical analysis. Therefore, different water table levels were employed under the same soil conditions, and these various levels caused changes in the thickness of the liquefied layer, which subsequently led to differences in the liquefaction-induced settlement. The correction value means that the referred percentage of the total soil depth is liquefied in the numerical analysis. To reduce the bias when estimating the value, the correction values employed were 0.23, 0.48, 0.8, and 1 for a water table that is 5 m, 3 m, 1 m, and 0 m beneath the ground surface, respectively (as shown in Fig. 12). Since the setting values of depth of ground water level in numerical analysis are 5 m, 3 m, 1 m, and 0 m, the ratios of the thickness of liquefied soil to the total thickness of soil layer are 23%, 48%, 80%, and 100% respectively. The viscosity determined using the viscosity chart is then divided by the correction value, and by doing so, a larger viscosity value is obtained, which means that the estimation of settlement will have a smaller value.

5.4. *Permeability effect*

The permeability of soil layers is not a constant, and varying permeability was thus adopted in the numerical analysis to consider its effect. Different soil permeabilities led to different thicknesses of the liquefied layer. The adopted soil permeability ranged from 10^{-3} to 10^{-5} in the numerical analysis, while the remaining soil conditions remained



(a) depth 3 m   (b) depth 4 m

Fig. 7. Excess pore water pressure in LIQCA.

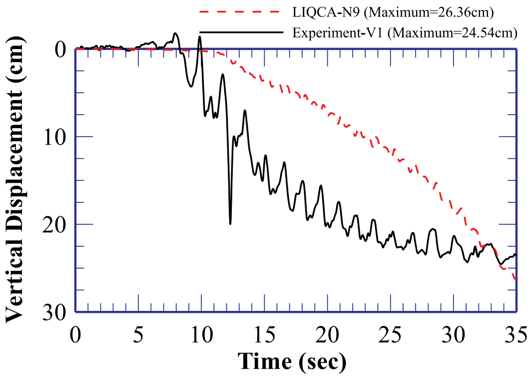


Fig. 8. Settlement in LIQCA

constant. The correction value means that the referred percentage of the total soil depth is liquefied in the numerical analysis, and the correction values were 0.23, 0.75 and 1 for permeability of 10^{-3}, 10^{-4} and 10^{-5} m/s, respectively (as shown in Fig. 12). Since the setting values of permeability in numerical analysis are 10^{-3}, 10^{-4}, 10^{-5}, and 10^{-6}, the ratios of the thickness of liquefied soil to the total thickness of soil layer are 23.2%, 75.6%, 100%, and 100% respectively. The viscosity found in the viscosity chart should then be divided by the correction value. A larger viscosity value is obtained using this step, which means that the estimation of settlement will have a smaller value. The results, which did not adopt any correction, are shown in Fig. 13. In Fig. 14, it shows the improved results after the four corrections have been employed in the estimation method, where the number of cases located in the 50%–200% zone is increased from 3 to 15 (a total of 17 cases). Therefore, this method is effective in reducing the bias for settlement in the estimation method.

In other words, corrected results are more consistent with experimental data than uncorrected results.

6. Comparison with other method

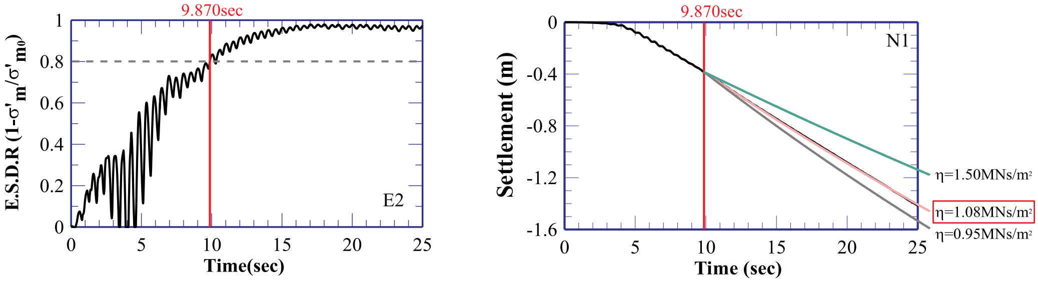
Shahir and Pak [1] proposed a practical formula that does not directly consider the parameters of soil behavior and seismic characteristics as follows,

$$\frac{S_f/Z_l}{Z_{l,m}^{0.5}\times q_{net}^{0.4}}=\left[0.0007\exp\left(-0.5\frac{B_f}{Z_l}\right)-0.0012\exp\left(-3.1\frac{B_f}{Z_l}\right)+0.0007\right]+0.0144\ln\left(\frac{H_t}{Z_l}\right), \tag{8}$$

where S_f is the average settlement of foundations; Z_l is the thickness of the liquefied layer; $Z_{l,m}$ is the maximum thickness of the liquefied layer; q_{net} is the net bearing capacity; B_f is the foundation width; and H_t is the total soil thickness. The performances using Shahir and Pak's formula are shown in Fig. 15. The result shows that more cases are located in the 50%–200% zone when using the equation of Sawicki and Mierczyński [11], and thus a more accurate estimation of liquefaction-induced settlement is provided.

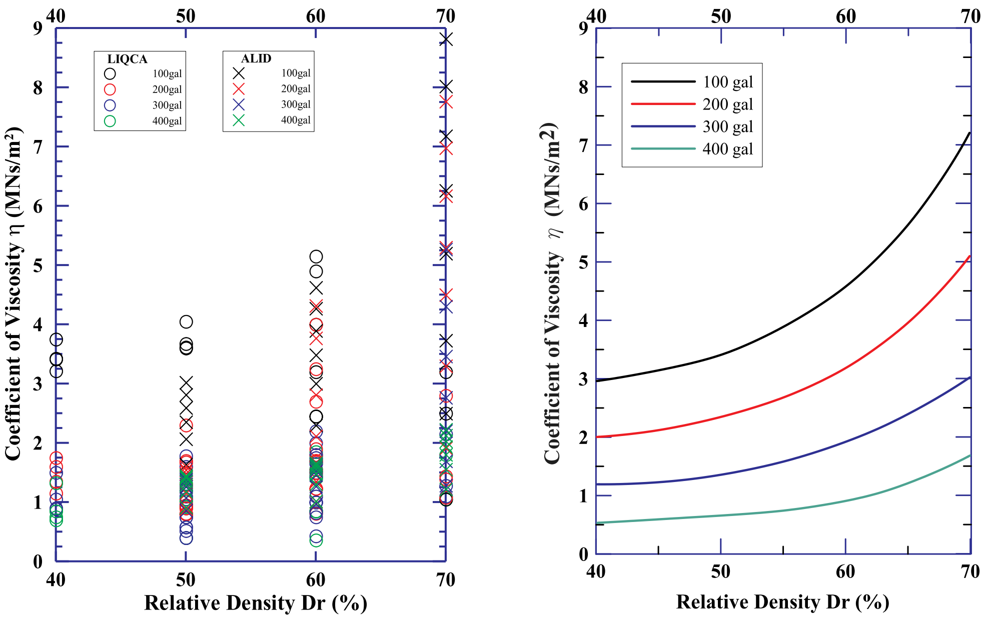
7. Summary and conclusion

The mechanism of inducing settlement is quite complicated. However, an appropriate simulation tool, a professional operator, an experienced expert, which is not always available. The residential area with high potential in liquefaction is large, and hard to evaluate the settlement one building by one. The object is to have the appropriate, and final value of the liquefaction-induced settlement. In this research, the simple method of evaluating the liquefaction-induced settlement was provided. With this method, a first-step evaluation of the liquefaction-



(a) ESDR versus time   (b) Settlement versus time

Fig. 9. Back calculation of viscosity.



(a)

(b)

Fig. 10. Viscosity chart obtained based on numerical cases.

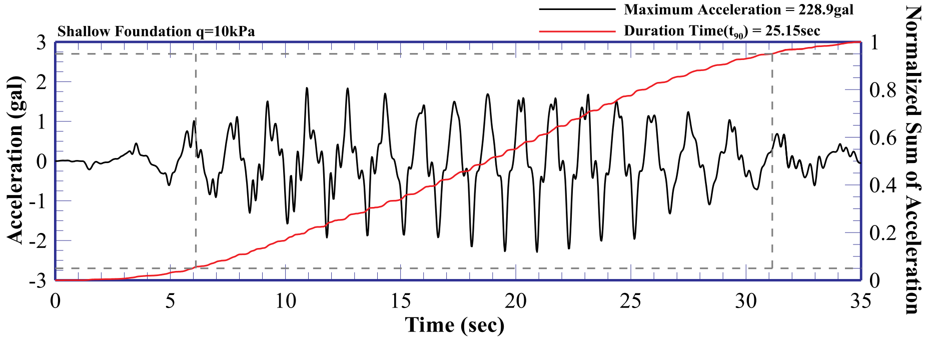


Fig. 11. Schematic diagram of T_{90} (case from Peng [14]).

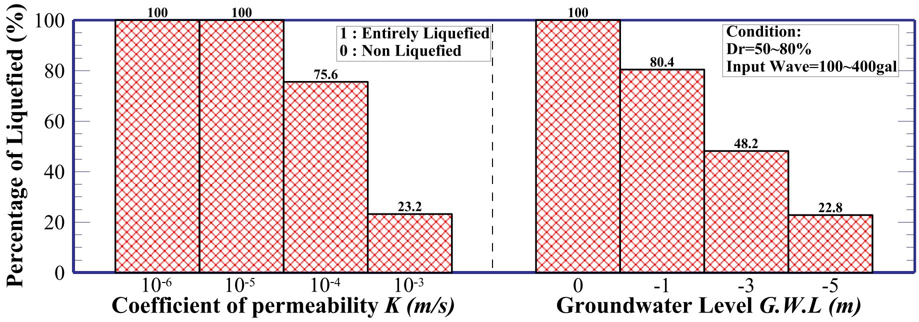


Fig. 12. Correction coefficient for ground water table and permeability+.

induced settlement could be obtained. Then, the situations of each building can be categorized into three categories, which are safe, need a further evaluation, dangerous. Then, the follow-up strategy in dealing with each category can be made.

Viscosity is affected by the dynamic behavior of soil, seismic activity, and soil-structure system interaction. In this study, viscosity was back calculated from the results of a numerical analysis based on the equation proposed by Sawicki and Mierczyński [12], and a viscosity chart was then compiled based on the results of 336 simulation cases. To accommodate different site conditions, four corrections were made. The representative viscosity can be obtained by the viscosity chart. By substituting the viscosity into (5), it was possible to estimate liquefaction-induced settlement. The estimated values were found to be similar to measured values obtained from experimental data.

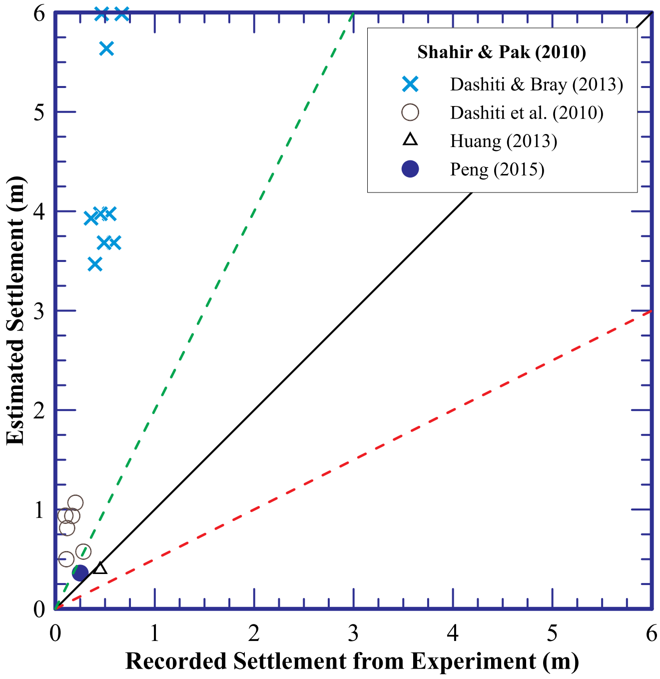


Fig. 13. Performances without corrections applied.

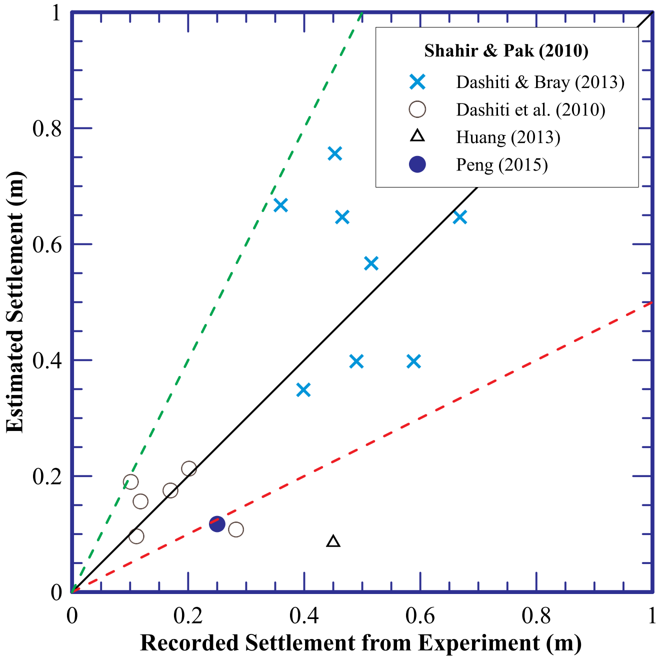


Fig. 14. Performances with corrections applied.

Six parameters are required in this estimation method: time duration, viscosity, moist unit weight of soil, foundation width, foundation length, and surcharge of structure. With intensity information obtained from near the region, the time duration and Amax can be obtained, and with the relative density of liquefiable soil and Amax, viscosity (which is a key parameter) can then be obtained from the viscosity chart. It is thus relatively easy to obtain information about the six parameters, and therefore simple to use this method to predict liquefaction-induced settlement.

Although this method provides an excellent performance, three outstanding issues need to be further addressed:

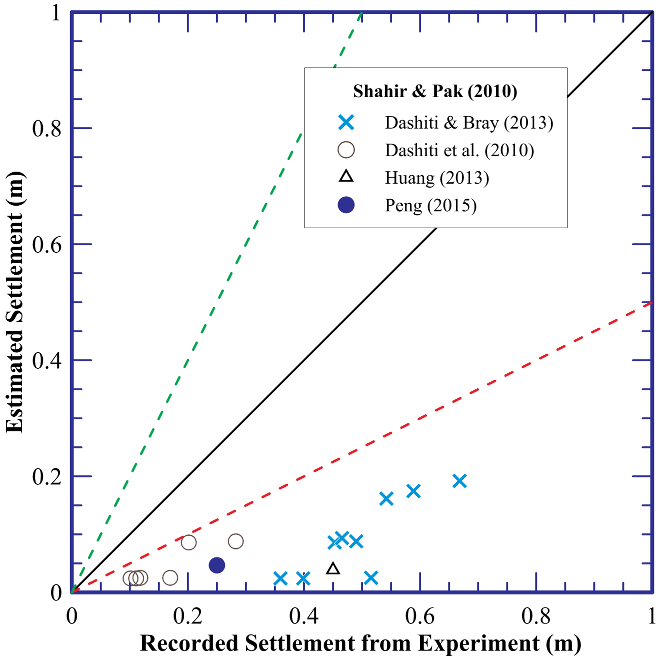


Fig. 15. Performance of Shahir and Pak's equation.

(1) No significant settlement occurs during the build-up process of excess pore water pressure.
(2) No inclination of the structure relating to shake, which is only limited to very small number of real cases.
(3) Viscosity is considered to be constant throughout the sinking process.

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