Calculation of the settlement of pile foundations taking into account the influence of soil liquefaction

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Abstract. Pile foundations are considered one of the most common foundation solutions for the construction of buildings and structures in difficult geotechnical conditions, including in seismic areas. However, during earthquakes specific processes often occur that negatively affect the interaction of piles with the surrounding soil. Soil liquefaction is one of the most difficult problems in geotechnics. Many historical cases show that buildings and structures on pile foundations are collapsed completely when the surrounding soil liquefies. One of the reasons is the increase in settlement of the foundation after soil liquefaction. Unlike the liquefaction potential assessment, or liquefaction-induced settlement of structures on shallow foundations, which are discussed by many researchers, the settlement of pile foundations after soil liquefaction has not been studied extensively. This paper presents the calculation of the settlement of pile foundations in different cases, with and without taking into account the influence of soil liquefaction, based on the hypothesis that the additional downward loads due to self-compaction of the liquefiable soil layers have an effect on piles. The analysis method in this study refers to the finite element method via PLAXIS 3D software. Finally, a brief comparison of the calculation results in the different considered cases is presented. The comparison shows how much the foundation settlement can increase if the influence of soil liquefaction is taken into account based on the suggested hypothesis. This paper provides a useful calculation and importance of further researches of liquefaction-induced settlement of pile foundations for seismic design.

1. Introduction
Earthquakes are well known as one of the causes of the worst damage to buildings and structures in the world. An earthquake can be accompanied by liquefaction of water-saturated sand and clay soils. Liquefaction is the process, in which the soil loses its bearing capacity and behaves like a dense liquid. According to many studies, soil layers can be partial or completely liquefied [1], depending on how much the pore water pressure in the soil increases. The increase in pore water pressure due to liquefaction in the soil is shown in figure 1.

Figure 1. Increase in pore water pressure in the soil due to liquefaction.
The high pore water pressure can lead to significant degradation of strength and stiffness of soil [2]. Soil liquefaction is one of the main causes of serious excessive damage to buildings and structures after earthquakes. Many of recent earthquakes such as the Niigata earthquake of 1964 [1], the Kobe earthquake of 1995 [3], the Armenian earthquake of 1988 and the Kamchatka earthquake of 2006 [4,5] have provided many examples of liquefaction induced damage.

Assessing the possibility of liquefaction of water-saturated soils during expected earthquakes and its possible consequences are the most important and complex engineering tasks for the construction of buildings and structures in seismic areas. The existing contribution to the study of liquefaction process and methods for assessing the possibility of soil liquefaction in seismic areas has been made by the studies of many scientists in the world such as Seed H.B. [6], Robertson P.K. [7], Voznesenskiy E.A. [8,9], Boulanger R.W., Idriss I.M. [10]. Those papers present determination of soil liquefaction following major earthquakes, the existing methods for its evaluation, such as seismic, dynamic and static sounding, also SPT and CPT methods.

Pile foundations are considered as an acceptable solution for foundations in liquefied soils, based on the analysis of their behavior in past earthquakes [6]. Nonetheless, there have been many historical cases where pile foundations have been collapsed completely due to the additional loads imparted by liquefied soil. The behaviour of pile foundations in liquefied soil during an earthquake is presented in several works of many researchers [2, 11-16]. Those papers present reviews of issues faced when designing pile foundation in seismic areas where exist liquefiable soils. Figure 2-3 shows two possible failure mechanisms of single pile due to soil liquefaction.

Figure 2. Bucking instability of single pile that rest in rock layer due to soil liquefaction [1].

Figure 3. Bearing failure of single pile that rest in a dense sand layer due to soil liquefaction [1].

Two possible mechanisms of failure for pile groups are shown in figure 4-5, in which the performance of pile groups is also accompanied by the lateral spreading of the ground [2,14].

Figure 4. Bearing failure of pile groups that rest in a rock layer due to soil liquefaction in laterally spreading ground [1].

Figure 5. Bearing failure of pile groups that rest in a dense sand layer due to soil liquefaction in laterally spreading ground [1].
Unlike the liquefaction potential assessment and the liquefaction-induced settlement of shallow foundations, which are discussed by many researchers [17-20], the settlement of pile foundations in liquefied soils has not been studied extensively. In this paper, the calculation of settlement of the pile foundation in the seismic area will be considered in different design cases, including the case where the influence of soil liquefaction is taken into account, based on the suggested hypothesis that the liquefied soil, after exposure to the seismic waves and liquefaction, pulls piles down, i.e. the additional downward loads due to self-compaction of the soil layers have an effect on piles.

2. Methods
In recent years, finite element method (FEM) is viewed as one of the most popular methods for solving geotechnical problems. In this paper the analysis method refers to FEM via PLAXIS 3D software.

Bearing capacity of a single friction pile, as known, can be defined as:

\[ F_d = F_{dR} + F_{df}, \]

where \( F_{dR} \) – end bearing capacity of the pile; \( F_{df} \) – skin resistance of the pile.

Bearing capacity of a single pile can be determined by the formulas in SP 24.13330.2011 [21] for calculating the settlement of the pile foundation. In this study, the bored piles foundation is considered in different design cases.

Case no. 1: The pile foundation is designed in normal geological engineering conditions.

In this case, according to SP 24.13330.2011 [21], bearing capacity of piles can be defined by the formula (2):

\[
\begin{align*}
F_{dR} &= \gamma_c \cdot \gamma_{eqR} \cdot R \cdot A, \\
F_{df} &= \gamma_c \cdot u \cdot \Sigma \gamma_{eqf} \cdot f_i \cdot h_i, \\
\end{align*}
\]

where \( \gamma_c \) – the coefficient of working conditions of piles equal to 1.0; \( \gamma_{eqR} \) and \( \gamma_{eqf} \) – accordingly the coefficient of working conditions of the soil at the pile tip and the surrounding soil that can be taken according to tables in SP 24.13330.2011 [21]; \( R \) – bearing capacity of the base soil at the pile tip, kPa; \( A \) – the cross-section area of the pile, \( m^2 \); \( f_i \) – skin resistance of the \( i \)-th layer of the surrounding soil, kPa; \( u \) – the cross-sectional perimeter of the pile, m; \( h_i \) – thickness of the \( i \)-th layer of the surrounding soil, m.

Case no. 2: The pile foundation is designed in the seismic area.

When calculating the bearing capacity of piles, the values of end bearing resistance and skin resistance of the pile should be multiplied by decreasing coefficients of the soil working conditions \( \gamma_{eq1} \) and \( \gamma_{eq2} \) [21,22]. The values of these coefficients depend on the type of soil and seismicity of the area. The formulas for the definition of bearing capacity in this case are presented below:

\[
\begin{align*}
F_{dR} &= \gamma_c \cdot \gamma_{eqR} \cdot \gamma_{eq1} \cdot R \cdot A, \\
F_{df} &= \gamma_c \cdot u \cdot \Sigma \gamma_{eqf} \cdot \gamma_{eq2} \cdot f_i \cdot h_i, \\
\end{align*}
\]

Moreover, the resistance of the soil in contact with the side surface of the pile is taken equal to zero to the depth \( h_d \), the definition of which is presented in SP 24.13330.2011[21]. According to this, the value of this depth \( h_d \) can be calculated as:

\[ h_d = \frac{a_3 \cdot (H + \alpha_e \cdot \alpha_s \cdot M)}{b_p \cdot \left( \frac{a_3}{\alpha_e} \cdot \gamma_1 \cdot \tan \phi + c_t \right)}, \]

where \( a_1, a_2, a_3 \) – dimensionless coefficients, which are taken equal to 1.5, 0.8, 0.6 for a single pile and the pile group that have the pile cap above the ground, 1.2, 1.2, 0 for the other cases; \( H \) – the value of horizontal force, kN; \( M \) – the value of bending moment, kN.m; \( b_p \) – conventional pile width, m; \( \alpha_e \)
– strain coefficient determined by formula (5); \( \gamma_i \) – the specific gravity of soil, kN/m\(^3\); \( \varphi_i \), \( c_i \) – the friction angle, degrees, and cohesion of soil, kPa.

\[
\alpha_e = \left( \frac{K \cdot b}{\gamma_i \cdot E \cdot I} \right)^{1/5}
\]

The value of \( h_d \) also must satisfy the following condition [21]:

\[
h_d \leq \frac{3}{\alpha_e}.
\]

Case no. 3: The pile foundation is designed is the seismic area where the influence of soil liquefaction is taken into account.

In this case, bearing capacity of piles as well can be calculated following the formula (3). Liquefaction-induced settlement of the pile foundation is determined based on the suggested hypothesis that the liquefied soil layers pull piles down due to self-compaction, herewith the soil layers to the depth \( h_d \) will not have zero volume strain in the direction of the z-axis.

3. Numerical simulation

3.1. The FEM Modelling

As a numerical example for the calculation of liquefaction-induced settlement, a bored piles foundation with massive circular piles is considered. Piles have a diameter of 1.5 m and a length of 33 m. The slab pile cap has a thickness of 2 m. The pile foundation is subjected to a 150 kN/m\(^2\) uniformly distributed load. The considered area in this example has a magnitude of 8.0 on the Richter magnitude scale. The displacement due to bodyweight is ignored. The properties of the pile foundation, which are used in Plaxis 3D software, are shown in tables 1-2.

| Parameter       | Name       | Pile foundation  | Unit    |
|-----------------|------------|------------------|---------|
| Young’s modulus | \( E \)    | \( 3 \cdot 10^7 \) | kN/m\(^2\) |
| Unit weight     | \( \gamma \) | 6                | kN/m\(^3\) |
| Beam type       | -          | Predefined       | -       |
| Predefined beam type | -     | Massive circular beam | - |
| Diameter        | \( \text{Diameter} \) | 1.5               | m       |
| Axial skin resistance | \( \text{Type} \) | Multi-linear     | -       |

| Parameter       | Name       | Slab pile cap  | Unit    |
|-----------------|------------|----------------|---------|
| Type of behaviour | \( \text{Type} \) | Elastic, isotropic | -     |
| Thickness       | \( d \)    | \( 2 \)        | m       |
| Weight          | \( \gamma \) | 15              | kN/m\(^3\) |
| Young’s modulus | \( E \)    | \( 3 \cdot 10^7 \) | kN/m\(^2\) |
| Poisson’s ratio | \( \nu \)  | 0.15            | -       |

The arrangement of piles in the foundation in this example is shown in figure 6, where \( d \) is the diameter of the pile. The pile spacing is 6\( d \), the distance from the axis of the pile to the pile cap edge is \( d \). The groundwater level is accepted at ground level.
Figure 6. The arrangement of piles in the foundation.
The design scheme of the model in PLAXIS 3D software with the pile foundation and the soil layer is presented in figure 7.

Figure 7. Design scheme of the model in Plaxis 3D software.
The ground in the design scheme includes 2 layers: layer 1 – silt with the thickness of 30 m, layer 2 – clay sand with the thickness of 10 m. The properties of the soil layers are given in table 3.

Table 3. Material properties for the soil layers.

| Parameter                  | Name   | Silt         | Clay sand    | Unit         |
|----------------------------|--------|--------------|--------------|--------------|
| Material model             | Model  | Hardening soil| Hardening soil| -            |
| Drainage type              | Type   | Undrained A  | Undrained A  | -            |
| Unit weight above phreatic level | $\gamma_{\text{unsat}}$ | 15           | 19.5         | kN/m$^3$     |
| Unit weight below phreatic level | $\gamma_{\text{sat}}$ | 15.2         | 20.3         | kN/m$^3$     |
| Young’s modulus            | $E_{\text{in0}}$ | 15000        | 50000        | kN/m$^2$     |
| Poisson’s ratio            | $\nu'$  | 0.35         | 0.3          | -            |
| Cohesion                   | $c'$   | 3.5          | 16           | kN/m$^2$     |
| Friction angle             | $\phi'$ | 23           | 39           | $^{\circ}$   |
| Dilatancy angle            | $\psi$ | 0            | 0            | $^{\circ}$   |
| Interface strength         | -      | Manual       | Manual       | -            |
| Interface reduction factor | $R_{\text{inter}}$ | 0.7          | 0.9          | -            |
| $K_0$ determination        | -      | Automatic    | Automatic    | -            |
The skin resistance of piles is defined by the Multi-linear option to take into account multiple soil layers with different properties and different resistances. In case no. 1, with all the soil conditions are shown above in table 3, according to the formulas (2), the values of end bearing resistance and the skin resistance of piles have been calculated equal $F_{dR} = 4258.8$ kN; $F_{d,f1} = 473.8$ kN for the first soil layer, $F_{d,f2} = 728.4$ kN for the second soil layer.

In case no. 2, after multiplying by decreasing coefficients of the soil working conditions $\gamma_{eq1} = 0.8$; $\gamma_{eq2} = 0.65$ for the first soil layer and $\gamma_{eq2} = 0.7$ for the second soil layer, the following values of bearing capacity of piles are obtained: $F_{dR} = 3407$ kN; $F_{d,f1} = 190.4$ kN for the first soil layer, $F_{d,f2} = 575.9$ kN for the second soil layer. The depth $h_d$, to which skin resistance of the pile is not taken into account, is taken equal to the maximum value according to the formula (6): $h_d = 11$ m.

For the calculation of liquefaction-induced settlement in case no. 3, the values of $F_{dR}$ and $F_{d,f}$ of piles in case no. 2 are used. Besides that based on the hypothesis that the soil layer pulls piles down due to self-compaction after liquefaction, the volume strain in z-direction is applied in PLAXIS 3D to the depth $h_d$ by a negative value to represents a shrinkage: $\varepsilon_{zz} = -1.000\%$.

3.2. Results of numerical simulation
The results of settlement in each of the examined cases are shown in the following figures 8-10.

![Figure 8. Colour shadings of the calculation in case no. 1.](image-url)
In figure 11 shown curvilinear dependencies of settlement of the pile foundation on the degree of applied load in 3 considered cases, where $\Sigma M_{\text{stage}} = 1.00$ tantamount 150 kN/m$^2$. 

Figure 9. Colour shadings of the calculation in case no. 2.

Figure 10. Colour shadings of the calculation in case no. 3.
Figure 11. Curvilinear dependencies of settlement of the pile foundation on the degree of applied load in different design cases ($\Sigma M_{\text{stage}} = 1.00$ tantamount 150 kN/m$^2$).

The results show a significant increase in settlement of the pile foundation. In detail, in case no. 1 the pile foundation under uniformly distributed load in normal geological engineering conditions has a settlement of 4.4 cm. In case no. 2, under the influence of the earthquake with a magnitude of 8.0 settlement of the foundation increased to 12.3 cm. In case no. 3 taking into account the influence of soil liquefaction, based on the suggested hypothesis, a settlement of 23 cm turned out, up to 87% in comparison to the result in case no. 2.

4. Conclusions

Settlement of structures on pile foundations following earthquakes induced soil liquefaction in an important area of research. A better understanding of the interaction between pile and liquefiable soil and the failure mechanisms of pile foundations is required.

In this paper, the calculation of settlement in different cases, with and without taking into account the seismic effect on pile foundations is considered. This paper also presents a suggested hypothesis about the influence of soil liquefaction on pile foundations and their settlement after an earthquake, that the liquefied soil, to a definite depth, pulls piles down due to self-compaction. The results of the calculation provided that the settlement in the foundation due to soil liquefaction based on the suggested hypothesis can significantly increase (up to 87%) in comparison to the conventional calculations, where the influence of soil liquefaction is not taken into account and can be not within acceptable tolerances.

Further researches of the liquefaction-induced settlement of pile foundations, including evidence of the accuracy of this suggested hypothesis are one of the main tasks in the design of structures in seismic areas for the reduction in the risk of the catastrophic destruction of foundations and structures during and after an earthquake.

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