Numerical Modelling of Shallow Foundation under Liquefaction

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Abstract: Failure of structures may force us to build a new structure. This may raise economic and environmental issues. Thus, building safe structures is essential for sustainable development. Liquification is one of the most critical geotechnical disasters that occurs in saturated sandy soils during earthquake or dynamic loading. To know the behavior of the liquefaction of soil, past studies were conducted analytically or experimentally. But this research is observing the soil liquefaction behavior numerically by conducting numerical modeling of vertical settlement, effective stress, and excess pore pressure, during earthquakes considering various soil and structural conditions utilizing Plaxis 2D FEM software. The calibration of the model was performed with a shear box test in the laboratory. Static shear stress phenomenon in liquefaction creates suction at mid of model and increases the effective stress at these specified locations. The load increases the generation of excess pore pressure.

Keywords: Liquefication, Shallow Foundation, Excess Pore Pressure, Effective Stress Analysis
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1. Introduction

The World contains extraordinarily engineered structures, few belong to archeological and historical backgrounds, and others are part of modern engineering. If we look at modern building construction, such as the construction of skyscrapers, the accountability of the safety and sustainability of such structures depends on modern-day engineers. Nonetheless, the safety parameter of such structures is susceptible to unanticipated catastrophes, which makes the research of such structures' behavior a vital necessity. Most of the structures of the modern world fail due to the soil mass instability. Liquefaction is one of the failures behind the instability of soil masses. Liquefaction-associated vertical settlements due to the seismic event result in hazardous damage and mighty destruction.

The soil behaves like a liquid while liquefaction occurs, which ultimately affects the safety of constructed buildings; similar conditions may occur during dynamic loadings such as cyclic loadings during seismic events or monotonic loadings, or due to another geo-mechanical phenomenon, i.e., pile driving or blasting. This soil behavior during such events makes the research a prerequisite for investigating whether it is susceptible or not.

When granular soil is subjected to shear loading of either a monotonic or cyclic nature, it has a natural tendency to condense, which can lead to liquefaction. The usual stress is transferred from the skeleton of soil to the water when the presence of water in pores prevents or reduces the amount of condensation that would otherwise occur. This can result in very high excess pore pressures, which ultimately leads to a very significant decrease in shear stiffness.

For a better understanding of the liquefication issue, numerous experimental experiments using centrifuge and 1-g shaking table tests were carried out. Significant amounts of negative pore pressure (EPWP) were produced beneath shallow foundations, according to the experimental study of Liu and Dobry Liu, [1] According to this method, Seed and Idriss H. Seed, Tokimatsu, Harder, & Chung, [2] H. B. Seed & Idriss, [3] arrived to the conclusion that soil liquefaction would take place if the FSL (presuming softly slopy ground with free-field environments) is less than or equal to 1. Since FSL is described as the correlation between cyclic stress ratio (CSR) and cyclic resistance ratio (CRR), it follows that the likelihood of soil liquefying is low if FSL is greater than 1.

The efficiency of stone columns on the seismic behavior of shallow foundations was explored by Adalier et al. Adalier, Elgamal, Meneses, & Baez, [4] in their experimental study. The collective role of shear strains given by post-liquefaction volumetric strains and superstructure as primary causes of total settlement of foundation were validated with centrifugal investigations Dashti & Bray; Dashti, Bray, [5] Pestana, Riemer, & Wilson, [6].

Physical model tests conducted by Yoo Yu, Zeng, Li, & Lian, [7] and Ebeido et al. Ebeido, Elgamal, & Zayed, [8] demonstrated that pile behavior is considerably manipulated by a variety of elements, including the resonance amid the superstructure and foundation, the force of inertia from the superstructure, the kinematic force from soil distortion, and the characteristics of input seismic motions.

It is narrated that the response of pore pressure and liquefaction while cyclic loading is dependent upon numerous factors, including length of time under sustained pressures, over consolidation ratio, sample preparation method, fines content, initial stress condition, initial void ratio, and confining pressure Chien, Oh, & Chang, [9]. In addition, cyclic triaxial testing is affected by nonuniformity in specimens, particularly at low confining pressure, as a result of sample preparation issues. As pore pressure develops in the sample of the cyclic triaxial test, soil particles tend to settle, triggering the lower portion to become denser and the upper portion to become looser. Nonuniform density results in nonuniform strain, which finally causes the upper segment of the sample to thin or neck. This nonuniformity can introduce significant ambiguity into the calculation of the application of cyclic triaxial test and liquefaction behavior outcomes Kramer, [10] Sitharam & Vinod, [11]. Because pore pressure and liquefaction origination are particularly perceptive to the aforementioned characteristics, it is critical to assess the behavior of the granular material using a numerical technique, i.e., the Finite Element Method under various conditions.

Calculating the overall stresses using geotechnical numerical models makes it possible to identify whether or not liquefaction takes place. When utilizing numerical models, one of the challenges that engineers have is determining how pore pressures are generated during an earthquake.

The computation of liquefaction-associated settlement of buildings is still a field of engineering exercise that needs improvement. It is anticipated that effective stress, nonlinear, dynamic, and fully coupled analysis will postulate useful insight into the SSI-induced forces and soil's nonlinear response Dashti & Bray, [5] Dashti [6]. The reaction of the ground and structure/foundation/ground systems has been extensively studied using finite element method modeling and other numerical simulation techniques. In this paper, the numerical modeling will be performed with UBC Sand model Plaxis 2D software to notice the changes in excess pore pressure (EPP) during dynamic loading that may induce large settlement in cohesionless soil under various foundations types, soil profile (position of bedrock), relative density of soil. The parameters of the numerical model were collected from field SPT and laboratory tests.

2. Research Methodology

Tsegaye [12] created the UBC3D-PLM model, which has been integrated into PLAXIS as a user-defined model. It comes primarily from the UBCSAND framework developed by Puebla et al. Puebla, Byrne, & Phillips, [13] Beaty and Byrne Beaty & Byrne, [14]. To anticipate liquefaction in sandy soils, the original UBCSAND model was created as a 2-dimensional model. It is formulated using a modified version of the Duncan-Chang method, that is grounded on classical plasticity theory and incorporates a hyperbolic strain hardening condition. The hardening rule establishes a connection amid mobilized friction angle and plastic shear strain under a stipulated load. This model takes into account the outcomes of shear-induced pore pressures during seismic events, that may lead to substantial variations in effective stress. The UBCSAND model provides pore water pressures in saturated components commensurate with laboratory test data, regardless of whether the soil is in a dilatant or contractive state. The model is based on an elastoplastic formulation in which the flow rule and hardening relationships are determined by variations in the stress ratio Stark, [15].

It has a non-associated plastic potential function and a Mohr-Coulomb yield surface in two dimensions. The well-known Mohr-Coulomb yield function is generalized in 3D primary stress space by the UBC3D-PLM model.

In order to comprehend how the method tackles the difficulty of 3D representation of yield surfaces, the whole set of Mohr-Coulomb yield functions is shown. Within the yield surface, the elastic behavior is ruled by a non-linear law. This nonlinear behavior is governed by two parameters: the elastic shear modulus (G) and the elastic bulk modulus (K). The following equations depict the connection between these two stress-dependent moduli.

$$K = K_B^e P_A \left(\frac{p}{P_{ref}}\right)^{me} \quad (1)$$

$$G = K_G^e P_A \left(\frac{p}{P_{ref}}\right)^{ne} \quad (2)$$

Where K^e_G and K^e_B are the moduli of shear and bulk, respectively, at a given stress level. Me and I are the stress exponent for G^e and B^e (around 0.5 for both). The parameters ne and I determine the rate of dependence of stiffness on stress. The pressure of atmospheric (PA=100 kPa) is typically used as the reference stress level (p_{ref}) in scientific literature. The model predicts only elastic behavior during the unloading process. Forasmuch as the stress point does not instantly return to the elastic zone, plastic behavior is anticipated once the stress state attains the yield surface. The model especially uses plastic hardening grounded on the principle of strain hardening. The hardening rule determines the amount of plastic strain (irreversible deformation) caused by shear strength mobilization. The Mohr-Coulomb model is modified by the UBCSAND stress-strain model to account for the plastic strains that occur across the loading process Doan, Park, & Lee, [16].

The angle of mobilized friction was calculated by using the Mohr-Coulomb yield criterion (3), shown as follows:

$$\sin\phi_{mob} = \frac{\sigma_1' - \sigma_3'}{\sigma_1' + \sigma_3'} = \frac{t_{mob}}{s'} \tag{3}$$

Where s' is the mean effective stress and t_{omb} is the mobilized shear stress.

In laboratory cyclic triaxial compression experiments, the stress-to-strain response for loading after the formation of large pore pressures manages to be driven by dilatation, resulting in the traditional banana-shaped or concave stress-strain loops. Byrne et al. P. Byrne, Park, & Beaty, [17] provide supplementary information on the constitutive model. The UBC3DPLM constitutive model addresses implicitly the undrained soil behavior. Therefore, the increase in pore pressure is calculated at each analytical step. Due to the lack of laboratory stress-strain statistics for the liquefiable soil, the UBCSAND model input parameters were mostly grounded on generic input values. These values offer an engineering approximation of stiffness, the production of pore water pressures as a result of post-liquefaction, and cyclic loading stress-strain behavior. The computed values for the designated parameters were derived using the available (N1) 60 blow counts in the coastal area of the study site.

3. Results and Discussions

It is possible to evaluate and present the outcomes of numerical investigation of the shallow foundation in various states in this section to comprehend the dynamic behavior of the foundation in liquefied soil. Plaxis 2D FEM software was used to understand the behavior of liquefiable soil under various soil and structural conditions. The width and depth of the model were 100 and 10 meters, and the width of the model did not interfere with structural boundary effects. The Plane strain model conditions were chosen and dynamic calculations were performed for 10 seconds. Two silty sand layers were used in this study as examples of liquefiable soil layers, and it was found that an acceptable N1(60) value of 15 provided the optimal input for the numerical investigation utilizing the UBCSAND model. The single N1(60) value can be used to directly generate each of the generic parameters needed for the UBCSAND model in Table 1. For PLAXIS analysis, the target earthquake (PLAXIS), and the input earthquake must be applied at the model's base.

Table:1 Model Parameters

$(N_1)_{60}$	Normalized, corrected SPT blow counts	Varies
ϕ_{cv}	Constant volume friction angle	33°
ϕ_f	Peak friction angle	$\phi_{cv} + (N_1)_{60}/10 + \max(0, ((N_1)_{60} - 15)/5)$
$k_G{}^e$	Elastic shear modulus number	$21.7 \times 20 \times (N_1)_{60}{}^{0.333}$
$k_B{}^e$	Elastic bulk modulus number	$0.7 \times k_G{}^e$
$k\ \ g^P$	Plastic shear modulus number	$k_G{}^e (N_1)_{60}{}^2 \times 0.003 + 100$
me	Elastic shear exponent	0.5
ne	Elastic bulk exponent	0.5
np	Plastic shear exponent	0.4
R_f	Failure ratio	$\min(1.1 \times (N_1)_{60}{}^{-0.15}, 0.99)$

Figure 1 shows the history of excess pore water pressure variations at various depths. The excess pore pressure (EPP) was 66 kPa at the top, whereas at the middle and bottom were 48 kPa and 20 kPa respectively. It suggests that liquefaction starts out close to the surface and progresses vertically downward.

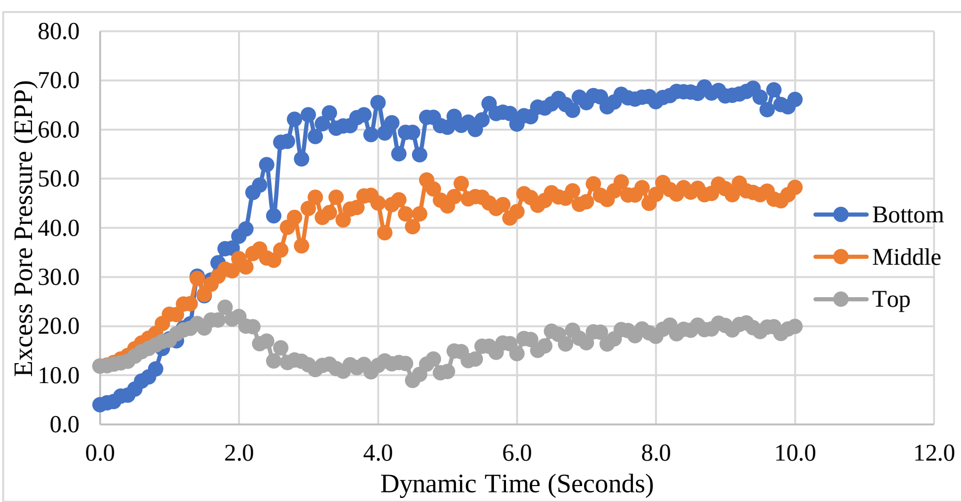

Figure 1: Excess Pore Water Pressure Variation at various depths

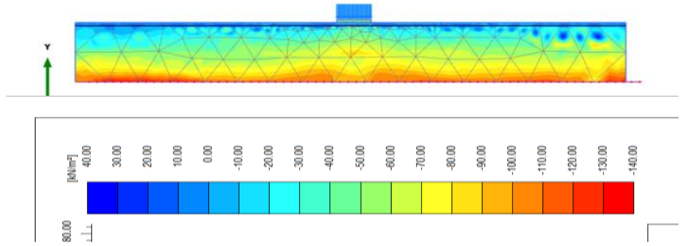

Figure 2: Excess pore water pressure at 12 kPa Load

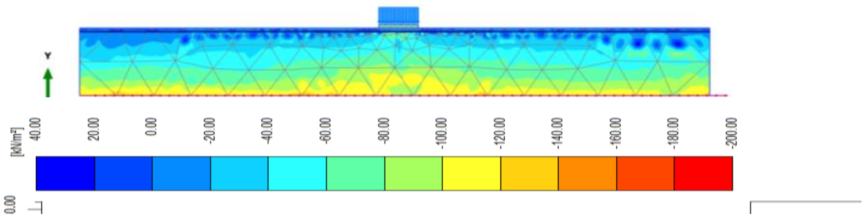

Figure 3: Excess pore pressure at 50 kPa Load

The irretrievable alteration in volumetric strain is calculated as a function of volumetric strain in the presence of a constant void ratio using the generation of pore pressure P. M. Byrne, [18]. Figure 2 and Figure 3 show the graphical representation of EPP in the mode below the foundation at the load of 12 and 50 kPa. The EPP variation with time (seconds) at the base of the model is given in Figure 4, and it was observed that the maximum EPP (Excess pore pressure) was 165 kPa at a load of 100 kPa and 66 kPa at the load of 12 kPa. Figure 5 shows the changes in effective stress with dynamic time at the bottom of the model, as depicted in Figure 5 the effective stress reduces to 10 kPa from 62 kPa. The pore pressure reduces the effective stress and thereby soil's shear strength decreases and large settlement may occur. Liquefaction reduces deformation resistance and causes high shear and volumetric strain to occur.

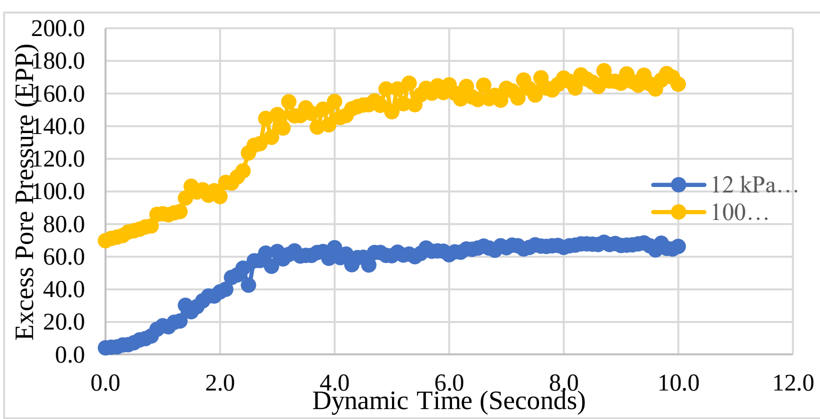

Figure 4: Effect of loading on excess pore pressure

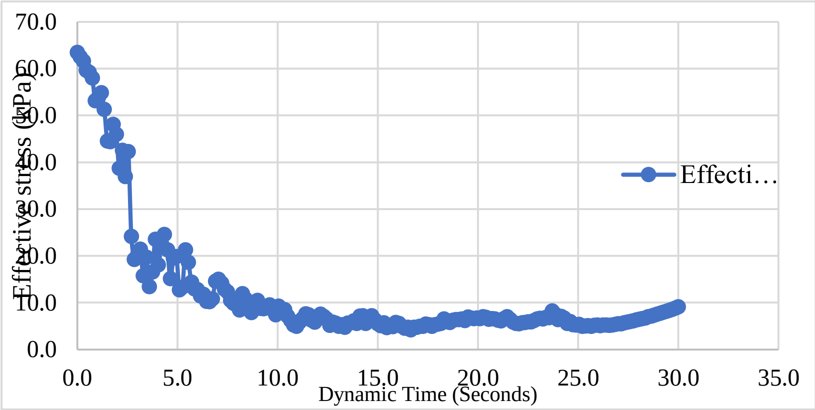

Figure 5: Effective stress variation with dynamic time

In the above analysis, the water table was placed at the top of the model to create the most favorable conditions for liquification. However, for determining the effect of the degree of saturation of soil on the settlement of soil under dynamic loading of the earthquake, the simulation was performed by changing the degree of saturation (DOS) of soil from 70 to 99 %. A large settlement of 120 mm was observed with the soil having a DOS of 99, whereas with the DOS of 60 and 80 % the settlements were 22 and 25 mm respectively. Considering Figure 6 the settlement shows a significant increase after DOS 90 %. Increasing liquefaction resistance is seen as a function of decreasing saturation degree. However, the majority of such studies utilize the Skempton coefficient B to calculate the sample's saturation level. When examining the influence of saturation on cyclic behavior, the use of B in laboratory work is suitable but unsatisfactory. It does not give a straightforward relationship between mechanical and physical soil parameters.

B is dependent on the compressibility of the pore air, whereas the presence of air in the soil causes suction, which hardens the material. Since suction and pore compressibility have opposing outcomes, it is inaccurate to analyze the liquefaction potential of unsaturated soil based solely on the volume compressibility of the soil skeleton. If air compressibility constituted the only distinction between unsaturated and saturated soil, usage of B as a saturation level reference would be appropriate.

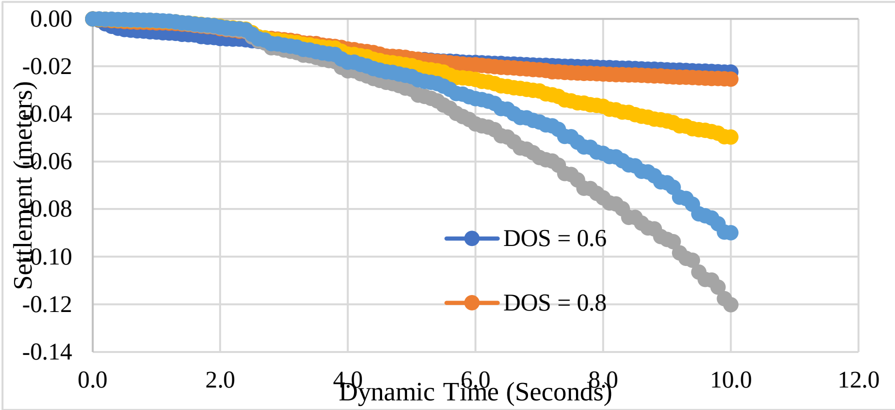

Figure 6: Variation of settlement and dynamic time with degree of saturation (DOS)

Figure 7 shows the variation of excess pore pressure with the change in degree of saturation (DOS). The DOS of 70 to 85 produces a small change in variation of excess pore pressure as compared to the degree of saturation (DOS) of soil to 100 %, and similar changes in settlement are observed in Figure 6. The initial increasing rate of pore pressure ratio is conditional on the initial degree of saturation according to the experimental study of Vernay et al. Vernay, Morvan, & Breul, [19].

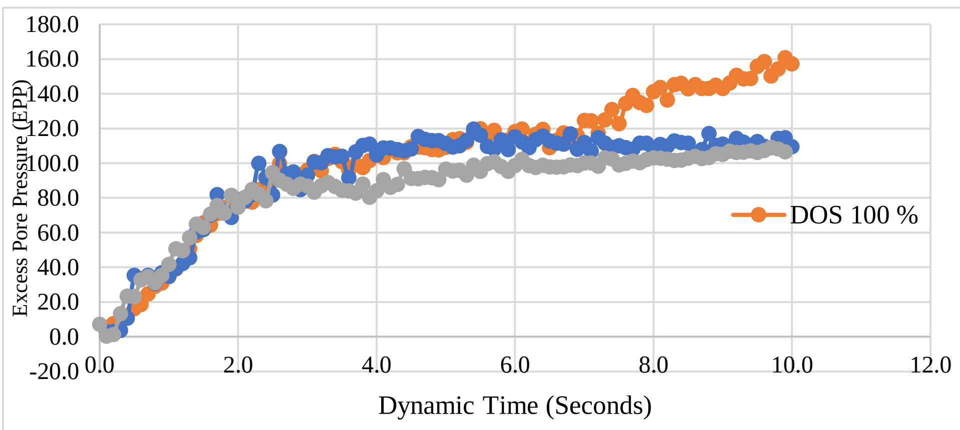

Figure 7: Effect of degree of saturation (DOS) of soil on excess pore pressure

4. Conclusions

In this paper, the numerical modeling was performed in Plaxis 2D with a user-defined UBCSAND-PLM model to observe the effect of excess pore water pressure generation and vertical settlements of isolated shallow foundations on liquifiable soil with dynamic analysis. In this research, various soil conditions were changed to understand the phenomenon of shallow foundation on liquefiable soil. From this study following conclusions may be drawn;

- In soft soil, during the dynamic analysis, the excess pore pressure generation decreases the effective stress of soil and thereby large vertical settlements below the isolated shallow foundation.
- The increase in the static load increases the generation of excess pore pressure.
- The degree of saturation of soil modeled with fully-coupled calculation in stagged construction. The degree of saturation of soil significantly affects excess pore pressure generation and vertical settlement of shallow foundations. The degree of saturation of soil more than 80% is critical to liquefication, as large excess pore pressure developed after 80 % of (DOS) of soil.
- In the isolated shallow foundation, the large suction developed below the footing due to static shear stress bias thereby increasing the liquefication resistance of the soil. However, no such observation is visualized in the free field of the model.

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