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EXTENDED ABSTRACT

Evaluation of the Nonlinear Response of a Single Pile Embedded in Sand Using Numerical Simulation

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1. Introduction

The performance of laterally loaded piles is one of the most critical issues in geotechnical engineering. The behavior of laterally loaded piles can be evaluated using 3D analysis. Although 3D analysis can accurately model the pile-soil interaction, it is used less than other methods (for example, the p-y approach, plane strain approach, etc.) due to increasing computational cost. In the literature, a comparison of 2D and 3D pile response with lateral loading has been studied. Boundary conditions of the pile head (free head and fixed head) and the ratio of buried length to the diameter (L/d) can significantly affect the accuracy of two-dimensional results. In the present study, the nonlinear performance of a single pile embedded in one layer of sand with a relative density of 40% has been evaluated using Abaqus v.6.14 under plane strain conditions and compared with experimental results. The primary objective of the present study is to enhance the precision of the two-dimensional model in predicting the pile response.

2. Methodology

To evaluate the accuracy of numerical modeling in predicting the seismic response of a single pile embedded in sand, the centrifuge test conducted by Gohl (1991) was used. In general, for the process of dynamic modeling, (1) proper definition of the interface element between the soil and structure, (2) consideration of Young's modulus variations with depth for the soil layer (or layers), (3) proper estimation of soil equivalent linear (EQL) modulus for dynamic analysis (for elastic-perfectly plastic constitutive models), (4) soil constitutive model, (5) type of numerical solution (i.e., explicit or implicit), (6) damping assigned to the model and, (7) boundary conditions used in the model have significant effects on simulation results, which have been evaluated in the following sections, respectively.

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2.1. Interface element

The present study used the thin layer element for the pile-soil interaction. The thickness of the thin-layer element was considered 100 mm (about 17% of the pile diameter). The peak friction angle of the interface element was determined based on Subba Rao et al. (1998) from Equation (1).

$$\frac{\delta_p}{\varphi_p} = 1 - 0.8exp(-15R^{0.54}) \tag{1}$$

Where δ_p is the peak friction angle of the interface element, ϕ_p is the peak friction angle of the sand, and R is the relative roughness determined from the following equation:

$$R = \frac{R_a}{D_{av}} \tag{2}$$

Where R_a is the average roughness, and D_{av} is the weighted average particle size of sand.

2.2. Consideration of Young's modulus variation with depth for soil layer

In the finite element model, the stiffness of the soil was considered to vary with depth. A common technique is dividing the desired soil layer into several sub-layers and assigning stiffness corresponding to the expected mean stress developed in the middle of each sub-layer. According to Hashash et al. (2020), for site response analysis conducted in the time domain, it is appropriate to divide the soil profile so that the maximum frequency of all sub-layers is the same and greater than 30 Hz. This methodology has been employed in the current study to divide the soil layer into multiple sub-layers.

2.3. Use of soil equivalent linear characteristics (EQL) for dynamic analysis (for elastic-perfectly plastic constitutive models)

Schnabel et al. (1972) suggested that the effective shear strain of the soil can be considered as 65% of the maximum shear strain (i.e., $\gamma_{eff} = 0.65\gamma_{max}$). According to Kagawa and Kraft (1980), the average shear strain of the soil around the pile can be expressed as a function of the relative lateral displacement of the pile. On the other hand, according to Yoshida et al. (2002), the effective shear strain of the soil can be determined from the zero-cross approach. In the present study, the accuracy of both approaches, proposed by Schnabel et al. (1972) and Yoshida et al. (2002), has been evaluated to determine the required Young's modulus for dynamic analysis.

2.4. Soil constitutive model

In this study, three constitutive models were used for Nevada sand: Mohr-Coulomb (MC), Drucker-Prager (DP), and Drucker-Prager/Cap (MDPC). It should be noted that the hydrostatic compression curves of Nevada sand, which are required in the MDPC model, were obtained based on the Vallejos (2008) model for different sub-layers. Plane strain condition is commonly used in geotechnical analyses. Therefore, the constitutive model parameters are often matched to obtain the same flow and failure response in plane strain conditions (Hibbitt et al., 2000). In the present study, the Drucker-Prager model parameters (*i.e.,* β, d, and ψ_{DP}) were matched with the Mohr-Coulomb model parameters (*i.e.,* ϕ, c, and ψ_{MC}) under the plane strain condition.

2.5. Type of numerical solution (Abaqus solvers)

Abaqus has two solvers, Abaqus/Standard, and Abaqus/Explicit. This study evaluated the accuracy of Abaqus solvers in predicting the pile-soil-superstructure system response.

2.6. Damping of the system

In the present study, the average value obtained from Darendeli (2001) and Menq (2003) was used to determine the small-strain damping of Nevada sand. Also, the combination of the mass and stiffness proportional damping was considered. In this way, the Rayleigh damping coefficients α and β were calculated for the two modes ω_i and ω_j, which showed the highest mass contribution from the frequency analysis. Rayleigh damping coefficients were considered to vary with depth.

2.7. Boundary conditions for dynamic analysis

In Abaqus software, infinite elements (INF) and multi-point constraints (MPCs) can be used to simulate dynamic boundary conditions. The present study evaluated the effect of lateral boundary conditions in two ways: using infinite elements along with MPC constraints (INF+MPC) and using only MPC constraints (MPC).

3. Results and discussion

Fig. 1 shows a comparison between the bending moment distribution obtained in the present study with the experimental results and the 3D analyses conducted by Wu and Finn (1997) and Rahmani et al. (2018). Based on this figure, considering the essential factors mentioned earlier, the 2D analysis method has accurately simulated the seismic response of the pile and provided reliable results.

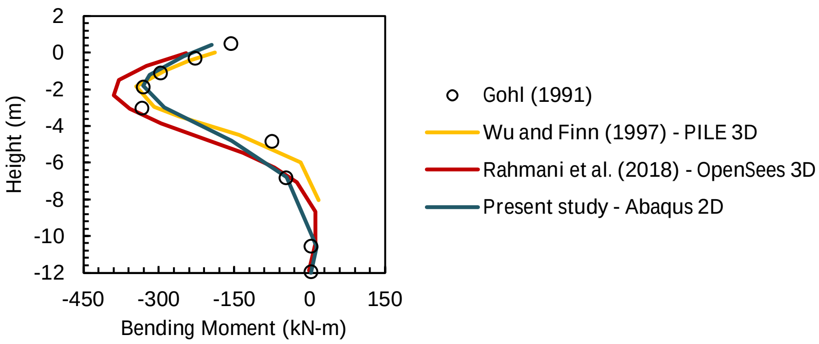

Fig. 1. Comparison of the measured and computed bending moment distribution for the single pile at $t=t_{d,max}$.

4. Conclusions

Based on the present study, it is recommended that in cases where acceleration time history is applied at the base of the model in Abaqus, infinite elements be used with MPC constraints instead of using infinite elements alone for lateral boundaries to overcome leakage of excitation energy. On the other hand, the simultaneous use of MPC constraints and infinite elements (*i.e.*, INF+MPC) has provided a suitable wave propagation simulation in the numerical model compared to the case where only MPC constraints are used for the lateral boundaries. The main drawback of the Mohr-Coulomb (MC) model for the INF+MPC case was an increase in model run time. Therefore, the Drucker-Prager (DP) model can be a suitable alternative to the MC model if its parameters are correctly matched under the plane strain condition. Based on the results, the thin-layer element could properly simulate the pile-soil interaction behavior. Accurately predicting effective shear strain is crucial for correctly estimating soil modulus in dynamic analysis. In finite element analysis employing elastic-perfectly plastic constitutive models, the zero-cross method is recommended to determine the effective shear strain in cases where the maximum shear strain exceeds 1%. Otherwise, Schnabel et al. (1972) approach can be used with acceptable accuracy. According to the results, the implicit solver has higher accuracy than the explicit solver in estimating the pile response. The implicit solver is recommended if the dynamic analysis has no convergence problem (or computational costs).