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Pile Group Effect Modeling and Parametric Sensitivity Analysis of Scoured Pile Group Bridge Foundations in Sandy Soils under Lateral Loads

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Abstract

Scour can increase the earthquake-induced damage in pile group foundations. Quantifying the parameter sensitivity of the seismic performance for scoured pile group foundations is essential for the optimal design and retrofit of bridges located in seismic-prone regions. Such quantification requires numerical models that are computationally efficient and accurate in describing the mechanical behavior associated with the complex soil-foundation-structure interaction of these systems. This study proposes an efficient finite element model (FEM) of pile groups based on a beam on the nonlinear Winkler foundation (BNWF) approach. This FEM uses asymmetric *p*-multipliers to describe the difference soil resistance exerted on leading and trailing piles when cyclic lateral loads are applied. The proposed FEM is validated through a comparison of the numerical response with the experimental measurements taken from a quasi-static test available in the literature, and is used to perform an extensive parametric sensitivity analysis to quantify the response sensitivity to eleven structural and soil parameters. Tornado diagrams are employed to identify an importance ranking of these parameters on the seismic performance of scoured pile groups. The obtained results indicate that the proposed FEM is able to capture both the global and local structural responses of pile group foundations. The parametric sensitivity analysis shows that pile group foundations have considerable ductility capacity. Pile diameter and axial load ratio of piles are the most important parameters for the seismic performance of pile groups. Increasing the pile diameter is the most efficient approach to improve the seismic performance of a pile group when considering scour effects. The seismic performance of a scoured pile group deteriorates for increasing piles' axial load ratio. For a deep pile group foundation, seismic performance is very little sensitive to pile length and relative density of sand. Based on the results of the parametric analysis, recommendations are proposed for the seismic design of pile group foundations with scour effects.

Keywords: Bridge scour; Pile group effect; Finite element model; Parameter sensitivity analysis; Tornado diagram; Ductility capacity; Soil-pile interaction.

Introduction

Reinforced concrete (RC) pile group foundations are widely used in bridge engineering practice (Fayyazi et al. 2012). In addition to bearing vertical loads, pile group foundations can be affected by significant lateral loads produced, e.g., by earthquake ground motions. Existing seismic design specifications often require that the pile group foundations remain in their elastic behavior range under design-level earthquake excitations, based on the capacity design philosophy (Mander et al. 1998), or allow for the formation of plastic hinges at the pile-cap connection (AASHTO 2022). However, pile damage is often unavoidable when strong earthquakes take place (Kawashima et al. 2009; Wei et al. 2008). In addition, scour has been reported as the main hazard causing bridge failures (Alipour and Shafei 2016; Capers et al. 2013; Wardhana and Hadipriono 2003), by exposing pile group foundations and reducing their lateral load capacity. Recent studies found that scour makes pile group foundations more prone to earthquake-induced damage than columns in a pile-supported bridge system (Wang et al. 2015, 2019, 2014). Therefore, the seismic design of scoured pile group foundations is a research topic of significant practical relevance. Extensive experiment- and/or simulation-based studies have focused on soil-pile interaction effects, including pile group effects and inertial/kinematic interaction of soil-pile-structure systems, in which the piles remained in the elastic range or exhibited a limited amount of plastic behavior (Boulanger et al. 1999; Brown et al. 1988; Hussien et al. 2016; Rollins et al. 2005). To investigate the seismic failure mechanism and ductility capacity of RC pile group foundations, a few experimental studies adopted cyclic static loads imposed on pile group foundations to simulate earthquake loadings (Banerjee et al. 1987; Chai and Hutchinson 2002; Liu et al. 2020; Park and Falconer 1983; Wang et al. 2016; Zhou et al. 2021a).

Significant research efforts have been devoted to develop numerical approaches for simulating and predicting the behavior of pile group foundations subject to lateral loads. The lateral response of piles is commonly analyzed using the beam on nonlinear Winkler foundation (BNWF) approach (Adeel et al. 2020; Allotey and El Naggar 2008; Heidari et al. 2014; Liu et al. 2020; Matlock and Ripperger 1956; Wang et al. 1998; Zhang and Hutchinson 2012), in which the soil-pile interaction is described using a *p*-*y* curve, where *p* denotes the soil resistance and *y* denotes the lateral displacement of the pile. Boulanger et al. (1999) developed a nonlinear constitutive model based on a combination of three components in series: (1) an elastic spring in parallel with a dashpot to model radiation damping, (2) a plastic spring, and (3) a gap component consisting of a nonlinear closure spring in parallel with a nonlinear drag spring. This material constitutive model was implemented as the uniaxial material denoted as *PySimple1* in the Open System for Earthquake Engineering Simulation (*OpenSees*) platform (McKenna 2011), and has been widely adopted by both the practicing and academic civil engineering community (Brandenberg et al. 2007; Hutchinson et al. 2004; Kramer et al. 2008). This BNWF-based soil pile interaction modeling approach has also been validated by a series of centrifuge tests (Boulanger et al. 2003), quasi-static tests (Hutchinson et al. 2005; Zhou et al. 2022b), and shake table tests (Shang et al. 2018). An aspect that has received significant attention for pile group foundations is the quantification and modeling of the so-called pile group effect (Brown et al. 1988), which corresponds to the reduction of the lateral capacity of a pile group with respect to the sum of the lateral capacities of the individual piles. This phenomenon is produced by the overlapping of the soil zones affected by the different piles, which is particularly evident for closely spaced pile groups. Due to the pile group effect, different pile rows provide different contributions to the overall lateral capacity of the pile group, with generally higher loads applied to the leading piles, which also tend to exhibit higher curvatures and higher ductility demands (Rollins et al. 2005; Wang et al. 2019; Zhou et al. 2021a). Therefore, the development of an accurate modeling method for the pile group effect is a critical issue in predicting the performance and the ductile behavior of pile group foundations subject to seismic and lateral cyclic loads. Brown et al. (1988) proposed the *p*-multiplier method to simulate the pile group effect. In this method, the lateral soil resistance of each pile row at a given embedded depth is described using a *p*-*y* spring, in which the load *p* is reduced by a load reduction factor, f_m (i.e., the *p*-multiplier). Different *p*-multipliers can be used for different pile rows, to reproduce the experimentally-measured effect of different contributions between leading and trailing piles, providing a simple and widely adopted approach in engineering practice (AASHTO 2020). However, for seismically excited pile groups, the loading direction continually changes during the seismic excitation, with piles frequently interchanging their condition between leading and trailing piles multiple times during any given seismic event. To address this issue, previous studies adopted an approximate approach using the average constant value of the *p*-multiplier for all piles in the group, called group efficiency factor or group reduction factor, to modify the soil resistance in front of the piles (Adeel et al. 2020; Brown et al. 2001; Lemnitzer et al. 2010; Liu et al. 2020). This modelling approach has been shown to provide accurate estimates of the global force-displacement response of a pile group (Lemnitzer et al. 2010; Liu et al. 2020). However, this uniform reduction factor cannot simulate the difference of lateral soil resistance among different rows in a pile group subjected to cyclic loads, which would be necessary to capture the curvature differences between piles in different rows.

In addition, the use of experimental testing can only investigate a limited set of physical and modeling parameter combinations, due to the high cost associated with each experimental sample. However, many different parameters can affect the behavior of pile group foundations, particularly when subjected to scour hazards (Blanco et al. 2019; Song et al. 2020). To mitigate this issue, numerical simulation based on finite element modeling (FEM) can be used to investigate the effects of parameters for which direct experimental testing is unfeasible. Blanco et al. (2019) carried out a numerical parametric pushover analysis on the ductile behavior of RC pile group foundations. However, their study was based on a limited number of parameters and, in particular, did not investigate (1) the effects of cyclic loading, (2) different pile group configurations beyond a 2 × 3 configuration, and (3) the relative importance of different parameters on the seismic performance of the pile group foundations. Therefore, a reliable finite element modeling approach is a necessary complement to experimental investigations in order to understand the effects and relative importance of the different parameters that are expected to affect the performance of scoured pile group foundations subject to seismic actions. In addition, an extended parametric sensitivity analysis would significantly help engineers identify the critical parameters for improving the seismic performance of pile foundations with scour potential.

This paper proposes a practical and straightforward FEM approach, based on a BNWF model with asymmetric *p*-multipliers, to simulate the soil-pile interaction of pile groups with multiple rows of piles subjected to cyclic loading from seismic excitation. The proposed model is validated through a comparison between the experimentally-measured and numerically-simulated global and local response of a scoured RC pile group foundation in sandy soil. This study also performs a detailed parametric sensitivity analysis based on the newly proposed numerical model to identify critical structural and/or soil parameters affecting the seismic performance of scoured pile group foundations and to determine the sensitivity rankings of these parameters.

Novelty and Relevance

This paper proposes for the first time an asymmetric *p*-multiplier to better model the differences between leading and trailing piles in RC pile group foundations with multiple pile rows in the direction of the loading and subject to cyclic loading. The proposed approach could be easily extended to other types of piles. This study also performs for the first time a comprehensive parametric sensitivity analysis for scoured RC pile group foundations subject to cyclic loading. The sensitivity analysis results are reported in terms of the effects on the piles' performance, particularly in terms of damage levels exhibited by the piles after cyclic loading. The results presented in this paper could represent the basis for future improvements in the design and seismic retrofit of pile group foundations under the combined effects of earthquake and scour.

Numerical Modeling and Validation

Description of Experimental Test

Zhou et al. (2021a) carried out a series of quasi-static tests on RC specimens of scoured 2×3 pile group foundations to investigate their ductile behavior during cyclic loading and their post-earthquake vertical load-carrying capacity under different lateral damage states. Their experimental data for specimen #3 (which was loaded to a maximum lateral displacement level of 100 mm) were used to validate the numerical model in this study. Figure 1 shows the test layout. The RC pile group consisted of six circular piles with a diameter D = 0.12 m and a length H = 4.30 m. The pile head was connected together by a concrete cap with a dimension of 1.50 × 1.00 × 0.60 m. The center to center spacing of adjacent piles was 0.36 m (i.e., 3D). The specimen was positioned in the center area of a container with an inside dimension of 3.10 (length) × 1.50 (width) × 4.20 m (height), and embedded in homogeneous sand with an average relative density D_r = 55%. The embedded depth was 3.70 m (30.83D), and the exposure length of each pile was equal to 0.60 m, representing a scour depth of 5D. Table 1 lists the property of the sand used in the test. The total initial axial force applied on the piles was equal to 85.4 kN, corresponding approximately to an axial load ratio $\eta =$ 5% for each pile. The axial load ratio is defined here as:

$$\eta=\frac{P}{f_c \cdot A_g} \tag{1}$$

where P is the axial (dead) load exerted on the individual pile, f_c denotes the peak strength of the unconfined concrete (with f_c = 25.20 MPa for this experimental test), and A_g is the pile gross cross-section area (with A_g = 0.0113 m^2 for this experimental test. The lateral load was provided by an actuator, identified as Actuator #1 in Figure 1a. Figure 2 presents the lateral loading protocol for specimen #3.

Figure 1b shows the steel reinforcement configuration for each pile. The longitudinal steel reinforcement ratio was 1.5% and was provided by six 6-mm-diameter longitudinal steel rebars. The core concrete of the piles was spirally confined by 3.5-mm-diameter galvanized-iron-wires (GIWs) with a center-to-center spacing of 35 mm, leading to a transverse reinforcement ratio of 1.215%. Tables 2 and 3 summarize the mechanical parameters of the concrete and steel used in the pile group specimen.

Selection of p-Multipliers for Pile Group Effect

The p-multipliers have been typically obtained from full- or small-scale experimental quasi-static or centrifuge test, or estimated using finite element analysis, often based on three-dimensional (3-D) models. Table 4 summarizes the values of the *p*-multipliers for pile groups in sandy soil reported in the literature. It is observed that the value of *p*-multiplier mainly depends on pile layout, pile-row location, and the ratio of pile spacing to diameter, as reported in previous studies (AASHTO 2020; Adeel et al. 2020), whereas the relative density of sand seems to have a relatively small effect. The value of f_m increases with the increase of pile center-to-center spacing S, and the *p*-multiplier for leading piles is generally larger than that for trailing piles. Based on the collected data of f_m listed in Table 4, the mean values of f_m for first, second, and third row piles of a three-row pile group with $S = 3D$ are 0.75, 0.41, and 0.33, respectively; and their standard deviations are 0.058, 0.034, and 0.055, respectively. For a two-row pile group with $S = 3D$, the mean values of the *p*-multiplier for the first and second row piles are 0.84 and 0.56, respectively; and their standard deviations are 0.048 and 0.082, respectively.

Figure 3 compares the *p*-multipliers for three-row pile groups obtained from the literature and reported in Table 4 with the values recommended by the AASHTO specifications as a function of the ratio S/D (AASHTO 2020). The values suggested by AASHTO refer to pile groups with three or more rows in the load direction; they are equal to 0.8, 0.4, and 0.3 for first, second, and third or higher row, respectively, when $S = 3D$, and to 1.0, 0.85, and 0.7, respectively, when $S = 5D$. A linear interpolation (shown in Figure 3) is used to determine the *p*-multiplier for pile spacing contained between $3D$ and $5D$. These values are found to be generally in good agreement with the *p*-multipliers obtained from the literature; thus, they are used in the modeling performed in this study to estimate the *p*-multipliers. For the three-row pile groups with $S = 2.5\ D$, the *p*-multipliers are taken equal to the mean values obtained from Table 4, i.e., f_m = 0.66, 0.38, and 0.29 for the first, second, and third row of piles, respectively. For the two-row pile groups with a pile spacing $S = 3D$, the *p*-multipliers are taken equal to the mean values obtained from Table 4, i.e., f_m = 0.84 and 0.56 for the first and second row of piles, respectively.

Finite Element Modelling

A finite element (FE) model of the pile group foundation in sandy soil is built based on the BNWF approach to simulate the quasi-static test previously described. Figure 4 illustrates the FE numerical model, which is developed and analyzed using the *OpenSees* software framework (McKenna 2011).

Modeling of piles and cap

The piles are modeled using displacement-based beam-column elements with distributed plasticity and fiber sections

(Barbato et al. 2010; Liu et al. 2020). An FE mesh convergence analysis was performed in this study to determine an appropriate FE discretization for the piles. This analysis indicated that the FEMs with pile element lengths equal to 0.5*D*, 1.0*D*, and 1.25*D* produce a converged (i.e., almost identical) response for both global and local response quantities. Therefore, each pile is discretized into FEs of length equal to 0.12 m (i.e., 1*D*), with the exception of the pile bottom element with a length of 0.22 m, as shown in Figure 4a. Each beam-column FE has five Gauss-Lobatto integration points. Different constitutive models are assigned to fibers corresponding to unconfined concrete, confined concrete, and longitudinal steel reinforcement. In particular, the axial stress-strain behavior of the concrete fibers is simulated using the uniaxial material *Concrete01*, which corresponds to the Kent-Scott-Park model with zero strength in tension (Scott et al. 1982). This model has been shown to properly represent the stress-strain behavior of GIW-confined concrete (Zhou et al. 2021b, 2022b). Figure 4c shows the backbone curves of this concrete model. The model parameters for confined and unconfined concrete are listed Table 2. The axial stress-strain behavior of the longitudinal steel reinforcement fibers is modeled using the uniaxial material *Steel02*, which corresponds to the Menegotto-Pinto model with kinematic and isotropic strain hardening (Filippou et al. 1983). The model parameters for the longitudinal steel reinforcement are given in Table 3. The pile cap is modeled using two elastic beam-column elements, and the cap bottom is connected with all six pile-heads by elastic beam-column elements. The axial and flexure stiffness of these elastic elements are set equal to 1,000 times that of the pile elements to simulate an approximatively rigid link between the cap and the pile heads. The gravity load corresponding to the self-weight of each pile element is applied to the corresponding nodes, and the cap weight is applied to the cap center. A constant vertical load is imposed on the cap-top node to produce an axial load ratio of 5% on each pile head section. All the degrees of freedom (DOFs) of the aboveground nodes are left unconstrained. For the belowground nodes, because the lateral loads were applied to the cap-center along the three-row pile direction only (i.e., along the global X-axis), the two translational DOFs in the XY plane are connected to zero-length elements representing the soil-pile interaction (which is described in the following section), the rotational DOF about the Z-axis is a free DOF, and the other three remaining DOFs (i.e., translation along the Z-axis and rotations about the X- and Y-axis) are fixed.

Modeling of soil-pile interaction considering pile group effect

This study introduces an innovative approach to model the pile group effect in soil-pile interaction systems subject to cyclic or dynamic loads. In fact, the approach commonly adopted in the literature describes the soil resistance to the lateral movement of a pile group through the use of a constant *p*-multiplier (i.e., the so-called group efficiency factor) equally applied to all piles of the group (Adeel et al. 2020; Brown et al. 2001; Lemnitzer et al. 2010; Liu et al. 2020). This group efficiency factor is commonly calculated as the average value of the *p*-multipliers for different pile rows, and is adopted because different piles alternate the roles of leading and trailing piles during cyclic loading or earthquake excitations. However, this constant reduction factor cannot correctly simulate the difference of lateral soil resistance among different rows in a pile group under cyclic loading, leading to inaccurate estimates of differential curvatures for piles in different rows.

This study proposes a new practical modeling method, which simulates the soil resistance in front of a pile at depth h by using two parallel springs consisting of (1) a common nonlinear *p-y* spring and (2) a nonlinear asymmetric spring, as illustrated in Figure 4e. In particular, the end nodes of each pile element below the ground surface are connected to the fixed nodes representing the soil site via zero-length elements. The load-displacement response of the zero-length element is described by a nonlinear *p-y* spring and a nonlinear asymmetric spring in parallel in the horizontal direction (X-axis), and by a nonlinear *t-z* spring in the vertical direction (Y-axis). The nonlinear *p-y* spring is modeled using the uniaxial material *PySimple1* in *OpenSees* (Boulanger et al. 1999), which is commonly used to describe the force-displacement relation for soils acting on piles. The backbone of the *p-y* constitutive model for sand is given as follows (API 2007; Chai and Song 2012):

$$p = A \cdot p_u \cdot \tanh\left(\frac{n_h \cdot h}{A \cdot p_u} \cdot y\right) \tag{2}$$

$$p_u = \min(p_{us}, p_{ud}) \tag{3}$$

$$p_{us} = (C_1 \cdot h + C_2 \cdot D) \cdot \gamma \cdot h \tag{4}$$

$$p_{ud} = C_3 \cdot D \cdot \gamma \cdot h \tag{5}$$

where p is the lateral resistance of soil at the embedded depth h; y denotes the lateral deflection of the pile at depth h; p_u is the ultimate resistance of the sand at depth h; A is a loading factor, which is equal to 0.9 for cyclic loading; p_{us} and p_{ud} denote the ultimate resistance of soil in the shallow and deep regions, respectively; n_h is the initial subgrade reaction modulus of sand, which can be obtained from API specification as a function of the sand friction angle; γ is the soil weight density; C_1, C_2, and C_3 are non-dimensional coefficients that depend on the effective friction angle (API 2007; Chai and Song 2012).

The nonlinear asymmetric spring is approximatively modeled using the uniaxial material *QzSimple1* in *OpenSees* (Boulanger et al. 1999), which has a behavior similar to the constitutive model for the *p-y* spring in the compression side but has an asymmetric and smaller soil strength in the tension side, as shown in Figure 4g. The parameters of the backbone q-z curve are adjusted to approximately reproduce the same backbone curve used for the p-y spring, whereas the suction factor is set equal to zero. This nonlinear asymmetric spring is used to model the asymmetric value of the p-multiplier of any given pile when a pile group is subject to cyclic or seismic loads, i.e., when two different p-multiplier values need to be applied to the same pile in a given row that is switching from leading (corresponding to the larger p-multiplier value, $f_{m,l}$) to trailing pile (corresponding to the smaller p-multiplier value, $f_{m,t}$) as the load changes direction. The asymmetric spring is oriented so that the compression side coincides with the side in which the pile is non-trailing. It is noted here that a single asymmetric p-y spring (with two different p-multipliers for the leading and trailing directions) could be used to produce the same behavior obtained through the combination of the symmetric p-y spring and the asymmetric q-z spring proposed in this study. However, such a constitutive model is not currently available in *OpenSees*. Thus, the soil resistances of the two lateral parallel springs are given as follows:

$$p_{sym}^{(n)} = f_{m,t}^{(n)} \cdot p \tag{6}$$

$$p_{asym}^{(n)} = (f_{m,l}^{(n)} - f_{m,t}^{(n)}) \cdot p \tag{7}$$

in which $p_{sym}^{(n)}$ denotes the soil resistance for the n-th row piles provided by the symmetric p-y spring; $p_{asym}^{(n)}$ denotes the soil resistance for the n-th row piles provided by the asymmetric spring; the superscript $n = 1, 2, \ldots, n_{\max}$ denotes the pile row number, and n_{max} is the total number of rows. As a result, the required input parameters of the *PySimple1* material in *OpenSees*, $p_{ult,}$ and y_{50} (Blanco et al. 2019), are given by:

$$p_{ult} = f_{m,t}^{(n)} \cdot p_u \cdot L_t \tag{8}$$

$$y_{50} = \frac{A \cdot p_u}{2n_h \cdot h} \cdot \ln\left(\frac{2A+1}{2A-1}\right) \tag{9}$$

in which p_{ult} denotes the ultimate soil resistance provided by the symmetric p-y spring; y_{50} denotes the soil displacement at 50% of p_{ult}; L_t denotes the tributary length of the soil-pile contact associated with the given node. The required input parameters of the *QzSimple1* material, $q_{ult,}$ and z_{50}, are given by:

$$q_{ult} = (f_{m,l}^{(n)} - f_{m,t}^{(n)}) \cdot p_u \cdot L_t \tag{10}$$

$$z_{50} = y_{50} \tag{11}$$

where q_{ult} denotes the ultimate soil resistance provided by the asymmetric q-z spring; z_{50} denotes the soil displacement at 50% of q_{ult}, respectively. It is noted here that, for cases in which the p-multiplier value for a pile row remains constant in the two opposite loading directions, the soil resistance corresponding to this pile row can be modeled more simply by using only the symmetric *p-y* springs with the appropriate value of the *p*-multiplier. For the numerical model of the quasi-static test considered in this study, which involves a pile group foundation with three rows of piles in the loading direction with a 3*D* pile spacing, the *p*-multiplier values are: $f_{m,l}^{(1)} = f_{m,l}^{(3)} = 0.8$, $f_{m,t}^{(1)} = f_{m,t}^{(3)} = 0.3$, and $f_{m}^{(2)} = f_{m,l}^{(2)} = f_{m,t}^{(2)} = 0.4$.

The vertical soil-pile friction behavior is simulated using *t-z* spring modeled with the *TzSimple1* material in *OpenSees* (Boulanger et al. 1999). The corresponding input parameters t_{ult} and z_{50} are given by (Mosher 1984):

$$t_{ult} = k_0 \cdot \gamma \cdot h \cdot \pi \cdot D \cdot L_t \cdot \tan(0.8\phi \cdot \pi / 180) \quad (12)$$

$$z_{50} = \frac{t_{ult}}{k \cdot \pi \cdot D \cdot L_t} \quad (13)$$

where t_{ult} is the ultimate friction force at the soil-pile interface within the tributary length L_t; k_0 is the coefficient of lateral earth pressure at rest and is set equal to 0.4; ϕ is the friction angle of sand; z_{50} is the displacement at which the friction force reaches 50% of t_{ult}; k denotes the initial tangent stiffness and can be expressed as a function of the friction angle (Mosher 1984). Finally, the FE model of the benchmark example used for validation describes the boundary conditions at the pile tips with vertical springs, the behavior of which is given by an ENT material with an initial stiffness of 1×10^7 kN/m, as shown in Figure 4h. The modeling parameter values for the quasi-static test used in Eqs. (2) through (13) are given in Table 5.

Numerical Model Validation

The proposed FEM approach for pile group effect modeling under cyclic loading, referred to as proposed cyclic model (PCM) hereinafter, is validated through a comparison with the experimental results available in Zhou et al. (2021a). In order to assess the performance of this approach with those commonly used in the literature, two additional FE models are built in *OpenSees*. The first additional FE model, referred to ordinary monotonic model (OMM), uses ordinary symmetric *p-y* springs with *p*-multipliers equal to 0.8, 0.4, and 0.3 for leading, middle, and trailing piles, respectively, and is subjected to a monotonic pushover analysis with a maximum lateral displacement equal to 100 mm. These *p*-multiplier values are equal to those recommended in AASHTO (2020) for a three-row pile group in sandy soil with a spacing of 3*D*. The second additional FE model, referred to as ordinary cyclic model (OCM), adopts a constant *p*-multiplier (i.e., group efficiency factor) equal to 0.5 applied to all piles of the group, and is subjected to a quasi-static cyclic loading. The value of the group efficiency factor is calculated as the average value of the *p*-multipliers for different pile rows, as recommended by Brown et al. (2001). All other modeling details are identical for the three considered FE models. The lateral load is applied to the cap center using displacement-controlled loading and the non-linear residual equations of equilibrium are solved using the Newton-Raphson algorithm with the command *Newton* in *OpenSees* (Mazzoni et al. 2006). It is observed here that all three modeling methods for the pile group effect are based on the BNWF model, which is commonly used in practical applications. In addition, the computational effort associated to both PCM and OCM is almost identical; in fact, the clock time for both cyclic analyses were approximately 1280 s and 1180 s, respectively, when using a personal computer with an Intel(R) Core (TM) i7-10750H CPU @ 2.60GHz and 32 GB RAM.

Figure 5 presents the global force-displacement curve comparisons between the experimentally-measured and numerically-simulated results. In particular, Figures 5a and 5b compare the global hysteretic force-displacement curves predicted by the PCM and OCM, respectively, with the experimental result; whereas Figure 5c compares the experimental backbone curve corresponding to positive displacements with the lateral force-displacement curve predicted by the three different numerical models used in this study. It is observed that both the PCM and the OCM provide an overall very good agreement with the cyclic experimental results. The comparison of the global lateral force-displacement backbone curve results indicate that the three FE models used in this study provide almost identical results in terms of global response quantities, which are in good agreement with the corresponding experimental results. This result (i.e., negligible differences in global response quantities) was expected because the three pile group effect modeling approaches used in this study differ only in the way the lateral soil resistance is distributed among different pile rows, but it has negligible effects on the overall soil resistance exerted on the entire pile group.

Figure 6 compares the experimentally-measured and numerically-predicted pile curvatures. As reported in Zhou et al. (2021a), the experimental values of the curvatures along the piles were obtained from strain gauges (identified by circles in Figure 6), with the exception of the values at the pile heads for displacement values of 30 mm and 50 mm, which were obtained from the data measured by linear potentiometers (identified by crosses in Figure 6). In order to compare consistent experimental and numerical values, the numerical values of the curvature were obtained as the cross-section curvatures at the location of the strain gauges for the experimental curvatures obtained by using the strain gauges; whereas they were obtained as the average curvature for the finite elements corresponding to the length of the linear potentiometers for the experimental curvature obtained by using the linear potentiometers. In order to quantify the accuracy of the different numerical results, Table 6 reports the normalized root mean squared error (Rizzo et al.

2018) for different displacement levels, which is calculated as follows:

$$\varepsilon_{FE} = \frac{\sqrt{\frac{1}{N}\sum_{i=1}^{N}(\phi_{FE,i} - \phi_{\exp,i})^2}}{\max\limits_{1\le i\le N}(|\phi_{\exp,i}|)} \tag{14}$$

where $\phi_{\exp,i}$ and $\phi_{FE,i}$ denote the experimentally-measured and numerically-predicted pile curvatures, respectively, at location $i = 1, 2, \ldots, N$ along the pile; N denotes the total number of the experimental curvature data points collected along a given pile; and FE = PCM, OCM, or OMM indicates the considered FE model. It is observed that the predicted pile curvatures by the PCM and OMM are almost identical (i.e., $|\varepsilon_{\text{PCM}} - \varepsilon_{\text{OMM}}| \le 2.6\%$ for all considered displacement levels and all pile rows), and present a good agreement with the experimental results (i.e., $2.1\% \le \varepsilon_{\text{PCM}} \le 10.2\%$ and $2.7\% \le \varepsilon_{\text{OMM}} \le 10.0\%$). When the pile group is subjected to cyclic loads, the PCM provides better predictions of the pile curvatures than the OCM, with $\varepsilon_{\text{PCM}} < \varepsilon_{\text{OCM}}$, except for leading piles at 10 mm displacement (for which $\varepsilon_{\text{PCM}} = 8.9\%$ and $\varepsilon_{\text{OCM}} = 5.6\%$). In particular, the OCM underestimates the curvature in the belowground plastic hinge regions and overestimates the embedded depth of the plastic hinge for the leading piles, whereas it overestimates the curvature in the belowground plastic hinge regions and underestimate the embedded depth of the plastic hinge for the trailing piles.

Parametric Sensitivity Analysis of Pile Group Foundations in Sandy Soils

Parametric Study Matrix

This study presents the results of an in-depth parametric analysis to quantify the seismic performance sensitivity of scoured pile group foundations. The parametric analysis is based the validated FE modeling method for pile group response under lateral loading and monotonic pushover analysis. Pushover analysis is preferred to a cyclic analysis to reduce the computational effort and because the validation results presented in the previous section of this paper show that both PCM and OMM produce practically identical global and local responses that are in good agreement with experimental results.

Eleven modeling parameters are investigated in this study: seven geometric parameters, i.e., pile configuration (*P.C.*), pile length (L_p), pile diameter (D), pile center-to-center spacing (S), scour depth (L_a), longitudinal steel reinforcement ratio (ρ_l), and transverse steel reinforcement ratio (ρ_s); one loading parameter, i.e., axial load ratio of the piles (η); two material parameters, i.e., concrete strength (f_c), and yield strength of the steel reinforcement (f_y); and one soil parameter, i.e., relative density of sand (D_r). A central composite design method is used to select the parameter combinations to be investigated, as shown in Table 7. For these combinations, C0 is selected as the central point, and every other combination is obtained by varying one parameter at a time, with three different values for each parameter. These parameter values are selected based on typical values encountered in practice and in the literature (Aviram et al. 2008; Blanco et al. 2019; Das 2002; Jones et al. 2002). As a result, 23 cases are considered in this study.

The modeling details and the constitutive model of the cover concrete and of the longitudinal steel rebars are identical to those used in the FE model of the quasi-static test. The elastic modulus and strain hardening ratio of the reinforcement were 201 GPa and 0.83% (Zhou et al. 2022a), respectively. In order to model steel reinforcement with spirals, the core concrete of the piles is modeled by using the uniaxial material *Concrete04* available in *OpenSees*, which corresponds to the Mander model with zero tension strength (Mander et al. 1988). Table 8 summarizes the values of the constitutive parameters used in the parametric study to model the core concrete. The modeling approach used here for the lateral and vertical soil-pile interaction is identical to that described for the OMM.

Ductility Development and Performance Limit States of Scoured Pile Groups

The piles in a scoured pile group forms a frame-like structure due to the constraints imposed by the surrounding soil and the pile-cap connection. Multiple plastic hinges can develop on the pile shafts, both above and below ground level, when they are subject to lateral loads (Liu et al. 2020; Wang et al. 2016; Zhou et al. 2021a). The plastic hinge development can be used to define limit states for both load and resistance factor design and performance-based design, as well as for assessing the safety and need for retrofit of damaged bridges after an earthquake event. Based on the structural behavior identified in the existing literature (Blanco et al. 2019; Liu et al. 2020; Wang et al. 2016; Zhou et al. 2021a), five performance limit states for a pile group are considered in this study: (1) first aboveground yielding (FAY), corresponding to the first yielding of any longitudinal steel reinforcement in the pile group, which for scoured piles is generally located at the interface between cap and leading piles; (2) first belowground yielding (FBY), corresponding to the easy-to-repair limit state introduced by Blanco et al. (2019), so called because the pile damage observed before reaching this level is limited to only the aboveground portion of the piles, which is easily accessible for post-earthquake inspection and repair; (3) peak lateral strength (PLS), beyond which lateral strength degradation initiates; (4) severe structural damage (SSD), which is identified by the core concrete crushing or the rupture of the longitudinal steel reinforcement in any pile within the pile group, whichever occurs first; and (5) ultimate residual strength (URS), defined here as a 15% reduction of the lateral strength from its peak value (Ataei and Padgett 2012; Shen et al. 2021). A multiple-level displacement ductility index for a pile group is defined here as:

$$\mu_i = \frac{\Delta_i}{\Delta_1} \tag{15}$$

where Δ_i represents the horizontal displacement of the cap center corresponding to any specific damage state i = 1, 2, 3, 4, and 5 (i.e., FAY, FBY, PLS, SSD, and URS limit states, respectively).

Figure 7 plots the backbone curves with markers to identify the displacements corresponding to the considered limit states for all 23 cases. For the 2 × 3 pile groups, the FAY and FBY occur on the leading piles, consistently with the findings of previous experimental studies (Liu et al. 2020; Zhou et al. 2021a); whereas the FAY and FBY occur on the trailing pile for the 2 × 2 pile group (i.e., case C1). The ductility development for the 3 × 3 pile groups is very similar to that for the 2 × 3 pile groups, with a lateral resistance equal to 1.5 times that of the corresponding 2 × 3 pile groups, as the pile group effect is identical for these two cases. For all the considered cases, the SSD corresponds to the condition of concrete crushing at the head of the leading piles, which occurs always before the rupture of the longitudinal steel reinforcement. The SSD limit state occurs always before the URS limit state, with the exception of cases C11 and C14. This phenomenon is observed because: (1) for C11, the low longitudinal steel reinforcement ratio induces a rapid post-peak strength reduction, thus reaching the URS limit state before the core concrete can reach its ultimate strain; and (2) for C14, the high transverse reinforcement ratio significantly increases the ultimate strain of the core concrete, thus delaying its crushing limit state.

The pile length has negligible impact on the lateral resistance of the pile group and on the displacements corresponding to the different limit states. It is noted that this conclusion is valid here because the piles behave as deep foundations without uplift. As expected, the lateral strength of a pile group significantly increases for increasing pile diameters. More specifically, the peak lateral strength of the studied pile groups is equal to 1511 kN, 7527 kN, and 18788 kN for D = 0.6 m, 1.2 m, and 1.8 m, respectively. The pile spacing has a slight effect on the peak lateral strength and the limit state displacements of a pile group. An increasing scour depth produces a reduction of the peak lateral strength of the pile groups and an increase of the displacement values corresponding to different limit states. In particular, the peak lateral strength of the pile groups are equal to 9988 kN, 7527 kN, and 5860 kN for a scour depth equal to 3D, 5D, and 7D (with D = 1.2 m), respectively. Increasing the longitudinal steel reinforcement ratio can enhance the lateral resistance of a pile group and increase the displacement corresponding to the first three limit states (FAY, FBY, and PLS). By contract, the transverse steel reinforcement ratio has a negligible effect on the global lateral load-displacement response of the pile group before PLS, but it becomes significant in controlling the strength degradation after peak. An increasing axial load ratio slightly increases the peak lateral strength of a pile group, but also accelerates the crushing of the core concrete and triggers a rapid degradation of the lateral strength, which is an undesirable behavior for structures subjected to earthquake excitations. The increase of concrete strength can improve the lateral strength of a pile group, but it results in a more rapid degradation of the lateral strength. By contrast, increasing the yield strength of steel reinforcement can delay the lateral strength degradation after peak, but it is not the most desirable approach for improving the lateral resistance of a pile group. The relative density of sand has very small effects on the peak lateral strength and the limit state displacements of a pile group.

Sensitivity Rankings of the Studied Parameters

The sensitivity rankings of the eleven parameters considered in this study in terms of seismic performance of a scoured pile group foundation is investigated by using the tornado diagram method (Barbato et al. 2010). The seismic performance of a scoured pile group is assessed with respect to three aspects, i.e.: (1) global resistance to an earthquake excitation; (2) residual displacement corresponding to the FBY limit state, which is directly related to the post-earthquake repair costs; and (3) ductility capacity. The parameter sensitivity for a given response R is quantified by the total relative swing, sw_R, which is defined as:

$$sw_R = sw_R^{(+)} + sw_R^{(-)} = \frac{\Delta R^{(+)}}{R_0} + \frac{\Delta R^{(-)}}{R_0} = \left|\frac{R^{(+)} - R_0}{R_0}\right| + \left|\frac{R^{(-)} - R_0}{R_0}\right| \qquad (16)$$

where R_0 is the value of the response quantity when the considered parameter is equal to the middle (reference) value; $R^{(-)}$ and $R^{(+)}$ denote the values of the response quantity when the considered parameter is assumed equal to the lower and upper bounds, respectively, which are given in Table 7; and $sw_R^{(-)} = \Delta R^{(-)} / R_0$ and $sw_R^{(+)} = \Delta R^{(+)} / R_0$ are the lower and upper relative swing, respectively, which are also referred to as one-side relative swings hereinafter. These calculated total relative swings are then sorted from high to low values and plotted from top to bottom to form the corresponding tornado diagram.

The global resistance of a pile group is quantified by three indexes, i.e.: (1) the lateral strength at the FAY limit state, referred to as yield strength of a pile group hereinafter, beyond which the yielding of a pile group initiates; (2) the strength enhancement coefficient of a pile group after yielding, denoted as *SE* and defined as follows:

$$SE = \frac{F_{\mathrm{PLS}}}{F_{\mathrm{FAY}}} \tag{17}$$

where F_{FAY} and F_{PLS} denote the lateral strength at FAY and PLS limit states, respectively, and which represents the capacity of a pile group after yielding; and (3) the normalized strength degradation rate after peak, *SDR*, which is proposed in this study for the first time and is defined as the ratio between the post-yield stiffness and the secant stiffness at yielding of a pile group as:

$$SDR = \frac{F_{\mathrm{PLS}} - F_{\mathrm{URS}}}{\Delta_{\mathrm{URS}} - \Delta_{\mathrm{FAY}}} \cdot \frac{\Delta_{\mathrm{FAY}}}{F_{\mathrm{FAY}}} = \frac{0.15 F_{\mathrm{PLS}} \cdot \Delta_{\mathrm{FAY}}}{(\Delta_{\mathrm{URS}} - \Delta_{\mathrm{FAY}}) \cdot F_{\mathrm{FAY}}} \tag{18}$$

where F_{URS} denote the lateral strength at the URS limit state, Δ_{FAY} and Δ_{URS} denote the displacement at the FAY and URS limit states, respectively.

Figure 8 shows the sensitivity rankings of the eleven parameters with respect to the global seismic resistance of the studied pile groups. The parameters below the horizontal dashed line have a small effect on the studied response with a one-side relative swing less than 0.05. It is observed that the yield strength of a pile group is most sensitive to the pile diameter (with a relative swing equal to 2.19), whereas strength enhancement coefficient and normalized strength degradation rate are most sensitive to the axial load ratio (with a relative swing equal to 0.19 and 0.85, respectively). Increasing the pile diameter can significantly improve the yield strength, raise the strength enhancement coefficient, and delay the strength degradation of a pile group. Increasing the axial load ratio slightly enhance the yield strength of a pile group, but it considerably accelerates the strength degradation and reduces the strength enhancement coefficient of a pile group, which is an undesirable effect for the scoured pile groups. The scour is also undesirable for a pile group, because it significantly weakens the pile group (e.g., the yield strength drops by 36% in this study when the scour depth increases from 3*D* to 7*D*), decreases the strength enhancement coefficient, and accelerates the strength degradation of a pile group. The transverse steel reinforcement ratio has a small effect on the yield strength and the strength enhancement coefficient of a pile group, but it can significantly delay the lateral strength degradation of a pile group after peak strength is reached (with a relative swing equal to 0.73). In addition, the global resistance of a pile group is mostly insensitive to the relative density of sandy soil and to the pile length. When compared with the yield strength and the normalized strength degradation rate, the strength enhancement coefficient is relatively stable and shows less sensitivity to the studied parameters, because its maximum one-side relative swing is only

0.12.

Figure 9 presents the parameter sensitivity ranking for the residual displacement of a pile group at FBY limit state (denoted as DR_{FBY} hereinafter). The residual displacement is obtained by the following two analysis steps through the use of the PCM: (1) load the model to determine the lateral displacement of the pile group at FBY limit state, and (2) unload the model until it reaches a zero-lateral force state. The displacement corresponding to the zero-lateral force state is regarded as the residual displacement of the pile group (Zhou et al. 2021a). As shown in Figure9, the top five parameters to which the residual displacement of a pile group at the FBY limit state is sensitive are (in decreasing order of importance): (1) axial load ratio, (2) yield strength of the steel reinforcement, (3) longitudinal steel reinforcement ratio, (4) pile diameter, and (5) scour depth in sequence, corresponding to the relative swings of 0.91, 0.54, 0.51, 0.49, and 0.26, respectively. Their increase leads to the increase of the residual displacement of a pile group, which is detrimental for the post-earthquake recovery efforts. The pile length and transverse steel reinforcement ratio affect less the residual displacement of a pile group corresponding to the FBY state since the one-side relative swing is less than 0.05.

Figure 10 shows the sensitivity rankings of the different parameters with respect to the displacement ductility of a pile group at the FBY, PLS, and SSD limit states. The top four parameters to which the FBY displacement ductility of a pile group is sensitive are (in decreasing order of importance) *P.C.*, scour depth, pile center to center spacing, and pile diameter. The other remaining seven parameters have a small effect on the FBY displacement ductility since the one-side relative swing is less than 0.05. The 2×3 and 3×3 pile groups present an almost identical FBY displacement ductility since they have the same pile group effect. The axial load ratio affects the most the displacement ductility of a pile group at the PLS and SSD states (with a relative swing equal to 0.55 and 1.18, respectively), but it has a very small impact on the ductility of a pile group at the FBY state (with a relative swing smaller than 0.006). The higher the axial load ratio causes a lower pile group ductility corresponding to PLS and SSD states. The increase of the scour depth reduces the ductility of a pile group at the FBY and PLS states, but it has almost no impact on the ductility of a pile group at the SSD state. Increasing the transverse steel reinforcement ratio improves the ductility of a pile group at the SSD limit state; however, the transverse steel reinforcement ratio has a negligible impact on the ductility of the pile group at the FBY and PLS limit states. The displacement ductility of a pile group is insensitive to the pile length and the relative density of sand.

Considerations for Seismic Performance Improvements

The parametric sensitivity analysis results can be used to provide meaningful recommendations for the seismic design and retrofit of pile group foundations in sandy soils subject to scour effects, as summarized in Table 9. In this table, the plus sign (+) indicates that increasing the corresponding parameter value improves the seismic performance of the pile group (i.e., having a positive effect); the cross mark (×) represents that the increase of the parameter value deteriorates the performance (i.e., having a negative effect); the horizontal line (—) indicates the response is not monotonically affected by the parameter; the slash mark (\) indicates that the corresponding parameter has a generally small effect on the seismic performance (i.e., both lower and upper relative swings are less than 0.05). In addition, the marks corresponding to the three parameters affecting the most each considered response are highlighted in bold, and the most important and second most important parameters for each considered response are identified by three and two marks, respectively.

In general, increasing the pile diameter is the most efficient measure to improve the seismic performance of a pile group when considering scour effects because it can significantly increase the yield strength and the strength enhancement coefficient of a pile group, and reduce the normalized strength degradation rate. Increasing the axial load ratio deteriorates the seismic performance of a scoured pile group because: (1) it increases its residual displacement at the FBY state, (2) accelerates the lateral strength degradation after peak strength, and (3) degrades the strength enhancement coefficient of the pile group. Increasing the center-to-center spacing between piles up to $5D$ has a small positive effect on the seismic performance of a scoured pile group; however, it requires to increase the cap size, resulting in an increase of construction costs. For a deep pile group foundation, the pile length and the relative density of sand have a negligible effect on the seismic performance of the pile group. The transverse reinforcement ratio has a small effect on a pile group's seismic performance before the peak lateral strength is achieved; however, the transverse steel reinforcement must be sufficient to prevent buckling of the longitudinal rebars, which becomes important for large inelastic behavior. The increase of the scour depth generally deteriorates the seismic performance of a pile group.

Conclusions

This study investigates the seismic performance of scoured pile group foundations. A new practical finite element modeling approach for the pile group effect is first proposed and validated by the experimental data available in the literature. The proposed approach can simulate the soil resistance difference among different pile rows in a pile group under lateral cyclic loads and provides more accurate local pile curvature results than the currently adopted methodology based on a constant pile group factor. It is also observed that both approaches provide almost identical results in terms of global response quantities, e.g., global lateral force-displacement response, which are in good agreement with existing experimental results. By using the validated model, an in-depth parametric analysis is performed to explore parameter sensitivity with respect to the seismic performance of a pile group foundation. The main findings of this parameter sensitivity analysis are as follows:

(1) Scour significantly weakens the seismic capacity of a pile group. It reduces the lateral strength and displacement ductility of a pile group, and increases the residual displacement of a pile group at the first belowground yielding (FBY) limit state (corresponding to an easy-to-repair condition).

(2) Increasing the pile diameter is the most effective measure to improve the seismic performance of a scoured pile group. Increasing the concrete strength, the yield strength of the steel reinforcement, and the longitudinal steel reinforcement ratio can increase the yield strength of a pile group. However, increasing the yield strength of the steel reinforcement and the longitudinal steel reinforcement ratio can also increase the residual displacement of a pile group at the FBY state. Higher axial load ratio slightly increases the lateral strength of the pile group foundation, but also significantly increases its residual displacement, reduces its lateral strength enhancement coefficient, and accelerates its lateral strength degradation. The transverse reinforcement ratio has negligible effects on the yield strength and the residual displacement at the FBY limit state of a pile group; however, a higher transverse reinforcement ratio can decrease the degradation rate of the lateral strength. For a deep foundation in medium dense sand, the pile length (for values higher than or equal to 30 times the pile diameter), pile spacing (for values smaller than or equal to five times the pile diameter), and relative density of sand (for values between 40% and 60%) have negligible effects on seismic performance of a scoured pile group foundation.

This study uses a set of quasi-static tests available in the literature to validate the proposed pile group effect modeling approach for pile group foundations subjected to lateral cyclic loads. It is recommended to validate this modeling approach also for pile group foundations subjected to dynamic conditions, e.g., seismic shakings. This study mainly focuses on scoured pile groups in a homogeneous sand subjected to lateral loads. Further studies are needed to quantify the impacts of different soil conditions and profiles. In addition, future numerical studies should investigate the use of incremental dynamic analysis to develop fragility curves and achieve a better understanding of the ductile behavior and the failure mechanisms of bridges with scoured pile group foundations. It is also suggested to investigate whether the p-multipliers vary with the depth and with the lateral deformation level of the pile group foundation, as well as how this potential variation affects the performance of a scoured pile group foundation.

Data Availability Statement

Some or all data, models, or code that support the findings of this study (including the section analysis executable codes and the force-displacement data for the pile group specimens) are available from the corresponding author upon reasonable request.

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Table 1. Sand parameters in the experimental test from Zhou et al. (2021a)

Parameter	Unit	Value
Unit weight (γ)	kN/m^3	15.95
Moisture content (w)	%	0.16
Average relative density (D_r)	%	55
Friction angle (ϕ)	Degree	33

Table 2. Mechanical properties of concrete from Zhou et al. (2021a)

Material	Peak strength (MPa)	Strain corresponding to peak strength	Strength at ultimate strain (MPa)	Ultimate strain
Unconfined concrete	25.20	0.0020	5.04	0.006
Confined concrete	29.05	0.0037	5.82	0.021

Table 3**.** Mechanical properties of steel reinforcement from Zhou et al. (2021a)

Material	Elastic modulus (MPa)	Yield strength (MPa)	Peak strength (MPa)	Strain corresponding to peak strength
ϕ6mm rebars	216,353	429	670	0.120
ϕ3.5mm GIW	135,441	317	421	0.148

Table 4. Values available in the literature of p-multipliers for pile groups with three or two rows in the load direction

Pile rows	References (Year)	Soil properties	Method used to estimate p-multipliers	Pile layout	D (m)	S/D	1st row	2nd row	3rd row
3	Brown et al. (1988)	Saturated medium dense sand (D_r = 50%) over stiff clay	Full-scale test	3×3	0.273	3	0.8	0.4	0.3
	Rollins et al. (2005)	Sand to silty sand ($D_r \approx$ 50%)	Full-scale test	3×3	0.324	3.29	0.8	0.4	0.4
			Numerical fitting	NS	NS	2.5	0.75	0.34	0.34
	Christensen (2006)	Medium to dense sand (ϕ = 35~40°) over soft clay and silt	Full-scale test	3×3	0.324	5.65	1	0.7	0.65
	McVay et al. (1995)	Medium loose sand (D_r = 33%)	1/45 scale centrifuge test	3×3	0.0095	3	0.65	0.45	0.35
		Medium dense sand (D_r = 55%)		3×3	0.0095	3	0.8	0.45	0.3
		Medium loose sand (D_r = 33%)		3×3	0.0095	5	1	0.85	0.7
		Medium dense sand (D_r = 55%)		3×3	0.0095	5	1	0.85	0.7
	McVay et al. (1998)	Medium loose (D_r = 36%) and medium dense sand (D_r = 55%)	1/45 scale centrifuge test	3×3	0.0095	3	0.8	0.4	0.3
	Kotthaus (1992)	Dense sand (D_r = 97%)	NS	1×3	--	3	0.75	0.42	0.45
				1×3	--	4	0.95	0.6	0.65
	Kim and Yoon (2011)	Dense sand (D_r = 73%)	Small-scale tests	1×3	0.012	4	0.85	0.6	0.45
		Medium dense sand (D_r = 55%)		3×3	0.012	3	0.7	0.35	0.3
	Vakili et.al (2020)	Loose sand (D_r = 39.5%)	Small-scale tests	1×3	0.02	2.5	0.7	0.44	0.29
				2×3	0.02	2.5	0.54	0.36	0.25
2	Reese et al. (2006)	NS	NS	2×2	NS	3	0.85	0.61	--
				2×2	NS	5	0.92	0.77	--
				1×2	NS	3	0.93	0.72	--
	Albusoda et al. (2018)	Loose sand (D_r = 32%) over 2 dense sand layers (D_r = 50% and 70%)	Small-scale tests	2×2	0.01	3	0.79	0.51	--
				2×2	0.01	6	0.88	0.72	--
			3-D FEM	2×2	0.01	3	0.81	0.5	--
				2×2	0.01	6	0.83	0.69	--
	Vakili et.al (2020)	Loose sand (D_r = 39.5%)	Small-scale tests	1×2	0.02	2.5	0.6	0.51	--
				1×2	0.02	3.5	0.88	0.61	--
	Abbas et.al. (2016)	Medium over loose and dense sand	3-D FEM	2×2	0.91	3	0.83	0.54	--

NS = not specified; -- = not present

Table 5. Modeling parameters for the sand soil in the benchmark example

Parameter	Unit	Value
C_1	None	2.49
C_2	None	3.10
C_3	None	41.73
γ	kN/m^3	15.95
n_h	kN/m^3	27145.0
k	kN/m^3	22620.0

Table 6. Normalized root mean squared error for pile curvatures at different lateral displacements

Displacement (mm)	FE	Pile location		
		Leading	Middle	Trailing
10	PCM	8.9%	10.2%	9.2%
	OCM	5.6%	12.4%	12.2%
	OMM	9.5%	10.0%	9.0%
30	PCM	8.4%	5.9%	2.1%
	OCM	10.0%	9.7%	6.9%
	OMM	8.2%	3.8%	4.7%
50	PCM	7.6%	7.3%	3.6%
	OCM	9.9%	8.9%	8.3%
	OMM	7.5%	6.8%	2.7%

Table 7**.** Parameter matrix and studied cases

Case ID	*P.C.*	L_p (D)	D (m)	S (D)	L_a (D)	ρ_l	ρ_s	η	f_c (Mpa)	f_y (MPa)	D_r (%)
C0	**2×3**	**35**	**1.2**	**3**	**5**	**0.016**	**0.012**	**0.2**	**50**	**420**	**50**
C1	**2×2**	35	1.2	3	5	0.016	0.012	0.2	50	420	50
C2	**3×3**	35	1.2	3	5	0.016	0.012	0.2	50	420	50
C3	2×3	**30**	1.2	3	5	0.016	0.012	0.2	50	420	50
C4	2×3	**40**	1.2	3	5	0.016	0.012	0.2	50	420	50
C5	2×3	35	**0.6**	3	5	0.016	0.012	0.2	50	420	50
C6	2×3	35	**1.8**	3	5	0.016	0.012	0.2	50	420	50
C7	2×3	35	1.2	**2.5**	5	0.016	0.012	0.2	50	420	50
C8	2×3	35	1.2	**5**	5	0.016	0.012	0.2	50	420	50
C9	2×3	35	1.2	3	**3**	0.016	0.012	0.2	50	420	50
C10	2×3	35	1.2	3	**7**	0.016	0.012	0.2	50	420	50
C11	2×3	35	1.2	3	5	**0.008**	0.012	0.2	50	420	50
C12	2×3	35	1.2	3	5	**0.024**	0.012	0.2	50	420	50
C13	2×3	35	1.2	3	5	0.016	**0.006**	0.2	50	420	50
C14	2×3	35	1.2	3	5	0.016	**0.018**	0.2	50	420	50
C15	2×3	35	1.2	3	5	0.016	0.012	**0.1**	50	420	50
C16	2×3	35	1.2	3	5	0.016	0.012	**0.3**	50	420	50
C17	2×3	35	1.2	3	5	0.016	0.012	0.2	**30**	420	50
C18	2×3	35	1.2	3	5	0.016	0.012	0.2	**70**	420	50
C19	2×3	35	1.2	3	5	0.016	0.012	0.2	50	**280**	50
C20	2×3	35	1.2	3	5	0.016	0.012	0.2	50	**520**	50
C21	2×3	35	1.2	3	5	0.016	0.012	0.2	50	420	**40**
C22	2×3	35	1.2	3	5	0.016	0.012	0.2	50	420	**60**

Table 8. Core concrete parameters used in the studied cases

Case ID	Peak strength (MPa)	Strain at peak strength	Ultimate strain
C0 - C12, C15, C16, C21, C22	64.9	0.00498	0.0167
C13	57.8	0.00357	0.0104
C14	71.3	0.00627	0.0231
C17	44.0	0.00667	0.0252
C18	85.3	0.00419	0.0131
C19	60.3	0.00406	0.0125
C20	68.0	0.00561	0.0197

Table 9. Seismic performance variation of a pile group by increasing the parameter value

Parameter	Description	Increasing yield strength	Increasing *SE*	Reducing *SDR*	Reducing DR_{FBY}	Increasing μ_{FBY}
P.C.	Pile configuration	++	—	—	—	———
L_p	Pile length	\	\	\	\	\
D	Pile diameter	+++	+	+	×	×
S	Pile center to center spacing	—	\	+	+	—
L_a	Scour depth	×	\	×	×	××
ρ_l	Longitudinal steel reinforcement ratio	+	+	×	×	\
ρ_s	Transverse reinforcement ratio	\	\	++	\	\
η	Axial load ratio	+	×××	×××	×××	\
f_c	Concrete strength	+	××	×	—	\
f_y	Yield strength of reinforcement	+	\	\	××	\
D_r	Relative density of sand soil	\	\	\	+	\

Figure 1**.** Quasi-static test overview (data from Zhou et al. 2021a): (a) schematic side view layout, (b) pile steel reinforcement, (c) full view of test, and (d) view of sand and aboveground piles (all units are in cm if not otherwise indicated).

Figure 2. Loading protocol for specimen #3 according to Zhou et al. (2021a).

Figure 3**.** *p*-multipliers for three-row pile groups in sand.

Figure 4**.** Numerical modeling: (a) schematic illustration of the entire model, (b) fiber section discretization of the piles, (c) concrete model, (d) steel model, (e) soil-pile interaction modeling, (f) *p*-*y* spring model, (g) asymmetric spring model, and (h) elastic-no-tension (ENT) spring model

Figure 5**.** Comparison of experimentally-measured and numerically-predicted lateral force-displacement results: (a) cyclic response predicted by PCM, (b) cyclic response predicted by OCM, and (b) backbone curve for positive displacements.

Figure 6. Comparison of experimentally-measured and numerically-predicted pile curvatures (Note: Exp. (s.g.) = experimental values obtained from strain gauges, Exp. (l.p.) = experimental values obtained from linear potentiometers.).

Figure 7. Sensitivity of lateral capacity to parameter variability: (a) pile configuration, (b) pile length, (c) pile diameter, (d) pile spacing, (e) scour depth, (f) longitudinal steel reinforcement ratio, (g) transverse reinforcement ratio, (h) axial load ratio, (i) concrete strength, (j) yield strength of reinforcements, and (k) soil relative density.

Figure 8. Parameter sensitivity rankings for the global resistance: (a) lateral strength at FAY limit state, (b) strength enhancement coefficient of a pile group after yielding, and (c) normalized strength degradation rate after peak.

Figure 9. Parameter sensitivity ranking for the residual displacement at FBY limit state.

Figure 10**.** Parameter sensitivity rankings for the displacement ductility at: (a) FBY, (b) PLS, and (c) SSD limit states.