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### Title

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

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11 **Abstract**

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

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

### 33 **Introduction**

34 Reinforced concrete (RC) pile group foundations are widely used in bridge engineering practice (Fayyazi et al. 2012).  
35 In addition to bearing vertical loads, pile group foundations can be affected by significant lateral loads produced, e.g.,  
36 by earthquake ground motions. Existing seismic design specifications often require that the pile group foundations  
37 remain in their elastic behavior range under design-level earthquake excitations, based on the capacity design  
38 philosophy (Mander et al. 1998), or allow for the formation of plastic hinges at the pile-cap connection (AASHTO  
39 2022). However, pile damage is often unavoidable when strong earthquakes take place (Kawashima et al. 2009; Wei et  
40 al. 2008). In addition, scour has been reported as the main hazard causing bridge failures (Alipour and Shafei 2016;  
41 Capers et al. 2013; Wardhana and Hadipriono 2003), by exposing pile group foundations and reducing their lateral load  
42 capacity. Recent studies found that scour makes pile group foundations more prone to earthquake-induced damage than  
43 columns in a pile-supported bridge system (Wang et al. 2015, 2019, 2014). Therefore, the seismic design of scoured  
44 pile group foundations is a research topic of significant practical relevance. Extensive experiment- and/or simulation-  
45 based studies have focused on soil-pile interaction effects, including pile group effects and inertial/kinematic interaction  
46 of soil-pile-structure systems, in which the piles remained in the elastic range or exhibited a limited amount of plastic  
47 behavior (Boulanger et al. 1999; Brown et al. 1988; Hussien et al. 2016; Rollins et al. 2005). To investigate the seismic  
48 failure mechanism and ductility capacity of RC pile group foundations, a few experimental studies adopted cyclic static  
49 loads imposed on pile group foundations to simulate earthquake loadings (Banerjee et al. 1987; Chai and Hutchinson  
50 2002; Liu et al. 2020; Park and Falconer 1983; Wang et al. 2016; Zhou et al. 2021a).

51 Significant research efforts have been devoted to develop numerical approaches for simulating and predicting the  
52 behavior of pile group foundations subject to lateral loads. The lateral response of piles is commonly analyzed using  
53 the beam on nonlinear Winkler foundation (BNWF) approach (Adeel et al. 2020; Allotey and El Naggar 2008; Heidari  
54 et al. 2014; Liu et al. 2020; Matlock and Ripperger 1956; Wang et al. 1998; Zhang and Hutchinson 2012), in which the  
55 soil-pile interaction is described using a  $p$ - $y$  curve, where  $p$  denotes the soil resistance and  $y$  denotes the lateral  
56 displacement of the pile. Boulanger et al. (1999) developed a nonlinear constitutive model based on a combination of  
57 three components in series: (1) an elastic spring in parallel with a dashpot to model radiation damping, (2) a plastic  
58 spring, and (3) a gap component consisting of a nonlinear closure spring in parallel with a nonlinear drag spring. This

59 material constitutive model was implemented as the uniaxial material denoted as *PySimple1* in the Open System for  
60 Earthquake Engineering Simulation (*OpenSees*) platform (McKenna 2011), and has been widely adopted by both the  
61 practicing and academic civil engineering community (Brandenberg et al. 2007; Hutchinson et al. 2004; Kramer et al.  
62 2008). This BNWF-based soil pile interaction modeling approach has also been validated by a series of centrifuge tests  
63 (Boulanger et al. 2003), quasi-static tests (Hutchinson et al. 2005; Zhou et al. 2022b), and shake table tests (Shang et  
64 al. 2018). An aspect that has received significant attention for pile group foundations is the quantification and modeling  
65 of the so-called pile group effect (Brown et al. 1988), which corresponds to the reduction of the lateral capacity of a  
66 pile group with respect to the sum of the lateral capacities of the individual piles. This phenomenon is produced by the  
67 overlapping of the soil zones affected by the different piles, which is particularly evident for closely spaced pile groups.  
68 Due to the pile group effect, different pile rows provide different contributions to the overall lateral capacity of the pile  
69 group, with generally higher loads applied to the leading piles, which also tend to exhibit higher curvatures and higher  
70 ductility demands (Rollins et al. 2005; Wang et al. 2019; Zhou et al. 2021a). Therefore, the development of an accurate  
71 modeling method for the pile group effect is a critical issue in predicting the performance and the ductile behavior of  
72 pile group foundations subject to seismic and lateral cyclic loads. Brown et al. (1988) proposed the  $p$ -multiplier method  
73 to simulate the pile group effect. In this method, the lateral soil resistance of each pile row at a given embedded depth  
74 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).  
75 Different  $p$ -multipliers can be used for different pile rows, to reproduce the experimentally-measured effect of different  
76 contributions between leading and trailing piles, providing a simple and widely adopted approach in engineering  
77 practice (AASHTO 2020). However, for seismically excited pile groups, the loading direction continually changes  
78 during the seismic excitation, with piles frequently interchanging their condition between leading and trailing piles  
79 multiple times during any given seismic event. To address this issue, previous studies adopted an approximate approach  
80 using the average constant value of the  $p$ -multiplier for all piles in the group, called group efficiency factor or group  
81 reduction factor, to modify the soil resistance in front of the piles (Adeel et al. 2020; Brown et al. 2001; Lemnitzer et  
82 al. 2010; Liu et al. 2020). This modelling approach has been shown to provide accurate estimates of the global force-  
83 displacement response of a pile group (Lemnitzer et al. 2010; Liu et al. 2020). However, this uniform reduction factor  
84 cannot simulate the difference of lateral soil resistance among different rows in a pile group subjected to cyclic loads,  
85 which would be necessary to capture the curvature differences between piles in different rows.

86 In addition, the use of experimental testing can only investigate a limited set of physical and modeling parameter

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

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

## 106 **Novelty and Relevance**

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

## 115 Numerical Modeling and Validation

### 116 *Description of Experimental Test*

117 Zhou et al. (2021a) carried out a series of quasi-static tests on RC specimens of scoured  $2 \times 3$  pile group foundations  
118 to investigate their ductile behavior during cyclic loading and their post-earthquake vertical load-carrying capacity  
119 under different lateral damage states. Their experimental data for specimen #3 (which was loaded to a maximum lateral  
120 displacement level of 100 mm) were used to validate the numerical model in this study. Figure 1 shows the test layout.  
121 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  
122 was connected together by a concrete cap with a dimension of  $1.50 \times 1.00 \times 0.60$  m. The center to center spacing of  
123 adjacent piles was 0.36 m (i.e.,  $3D$ ). The specimen was positioned in the center area of a container with an inside  
124 dimension of 3.10 (length)  $\times$  1.50 (width)  $\times$  4.20 m (height), and embedded in homogeneous sand with an average  
125 relative density  $D_r = 55\%$ . The embedded depth was 3.70 m ( $30.83D$ ), and the exposure length of each pile was equal  
126 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  
127 force applied on the piles was equal to 85.4 kN, corresponding approximately to an axial load ratio  $\eta = 5\%$  for each  
128 pile. The axial load ratio is defined here as:

$$\eta = \frac{P}{f_c \cdot A_g} \quad (1)$$

129 where  $P$  is the axial (dead) load exerted on the individual pile,  $f_c$  denotes the peak strength of the unconfined concrete  
130 (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<sup>2</sup> for this  
131 experimental test. The lateral load was provided by an actuator, identified as Actuator #1 in Figure 1a. Figure 2 presents  
132 the lateral loading protocol for specimen #3.

133 Figure 1b shows the steel reinforcement configuration for each pile. The longitudinal steel reinforcement ratio was  
134 1.5% and was provided by six 6-mm-diameter longitudinal steel rebars. The core concrete of the piles was spirally  
135 confined by 3.5-mm-diameter galvanized-iron-wires (GIWs) with a center-to-center spacing of 35 mm, leading to a  
136 transverse reinforcement ratio of 1.215%. Tables 2 and 3 summarize the mechanical parameters of the concrete and  
137 steel used in the pile group specimen.

### 138 *Selection of $p$ -Multipliers for Pile Group Effect*

139 The  $p$ -multipliers have been typically obtained from full- or small-scale experimental quasi-static or centrifuge test, or

140 estimated using finite element analysis, often based on three-dimensional (3-D) models. Table 4 summarizes the values  
141 of the  $p$ -multipliers for pile groups in sandy soil reported in the literature. It is observed that the value of  $p$ -multiplier  
142 mainly depends on pile layout, pile-row location, and the ratio of pile spacing to diameter, as reported in previous  
143 studies (AASHTO 2020; Adeel et al. 2020), whereas the relative density of sand seems to have a relatively small effect.  
144 The value of  $f_m$  increases with the increase of pile center-to-center spacing  $S$ , and the  $p$ -multiplier for leading piles is  
145 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  
146 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  
147 standard deviations are 0.058, 0.034, and 0.055, respectively. For a two-row pile group with  $S = 3D$ , the mean values  
148 of the  $p$ -multiplier for the first and second row piles are 0.84 and 0.56, respectively; and their standard deviations are  
149 0.048 and 0.082, respectively.

150 Figure 3 compares the  $p$ -multipliers for three-row pile groups obtained from the literature and reported in Table 4  
151 with the values recommended by the AASHTO specifications as a function of the ratio  $S/D$  (AASHTO 2020). The  
152 values suggested by AASHTO refer to pile groups with three or more rows in the load direction; they are equal to 0.8,  
153 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,  
154 when  $S = 5D$ . A linear interpolation (shown in Figure 3) is used to determine the  $p$ -multiplier for pile spacing contained  
155 between  $3D$  and  $5D$ . These values are found to be generally in good agreement with the  $p$ -multipliers obtained from  
156 the literature; thus, they are used in the modeling performed in this study to estimate the  $p$ -multipliers. For the three-  
157 row pile groups with  $S = 2.5 D$ , the  $p$ -multipliers are taken equal to the mean values obtained from Table 4, i.e.,  $f_m =$   
158 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  
159 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  
160 the first and second row of piles, respectively.

### 161 ***Finite Element Modelling***

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

### 165 **Modeling of piles and cap**

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

167 (Barbato et al. 2010; Liu et al. 2020). An FE mesh convergence analysis was performed in this study to determine an  
168 appropriate FE discretization for the piles. This analysis indicated that the FEMs with pile element lengths equal to  
169  $0.5D$ ,  $1.0D$ , and  $1.25D$  produce a converged (i.e., almost identical) response for both global and local response  
170 quantities. Therefore, each pile is discretized into FEs of length equal to 0.12 m (i.e.,  $1D$ ), with the exception of the  
171 pile bottom element with a length of 0.22 m, as shown in Figure 4a. Each beam-column FE has five Gauss-Lobatto  
172 integration points. Different constitutive models are assigned to fibers corresponding to unconfined concrete, confined  
173 concrete, and longitudinal steel reinforcement. In particular, the axial stress-strain behavior of the concrete fibers is  
174 simulated using the uniaxial material *Concrete01*, which corresponds to the Kent-Scott-Park model with zero strength  
175 in tension (Scott et al. 1982). This model has been shown to properly represent the stress-strain behavior of GIW-  
176 confined concrete (Zhou et al. 2021b, 2022b). Figure 4c shows the backbone curves of this concrete model. The model  
177 parameters for confined and unconfined concrete are listed Table 2. The axial stress-strain behavior of the longitudinal  
178 steel reinforcement fibers is modeled using the uniaxial material *Steel02*, which corresponds to the Menegotto-Pinto  
179 model with kinematic and isotropic strain hardening (Filippou et al. 1983). The model parameters for the longitudinal  
180 steel reinforcement are given in Table 3. The pile cap is modeled using two elastic beam-column elements, and the cap  
181 bottom is connected with all six pile-heads by elastic beam-column elements. The axial and flexure stiffness of these  
182 elastic elements are set equal to 1,000 times that of the pile elements to simulate an approximately rigid link between  
183 the cap and the pile heads. The gravity load corresponding to the self-weight of each pile element is applied to the  
184 corresponding nodes, and the cap weight is applied to the cap center. A constant vertical load is imposed on the cap-  
185 top node to produce an axial load ratio of 5% on each pile head section. All the degrees of freedom (DOFs) of the  
186 aboveground nodes are left unconstrained. For the belowground nodes, because the lateral loads were applied to the  
187 cap-center along the three-row pile direction only (i.e., along the global X-axis), the two translational DOFs in the XY  
188 plane are connected to zero-length elements representing the soil-pile interaction (which is described in the following  
189 section), the rotational DOF about the Z-axis is a free DOF, and the other three remaining DOFs (i.e., translation along  
190 the Z-axis and rotations about the X- and Y-axis) are fixed.

### 191 **Modeling of soil-pile interaction considering pile group effect**

192 This study introduces an innovative approach to model the pile group effect in soil-pile interaction systems subject to  
193 cyclic or dynamic loads. In fact, the approach commonly adopted in the literature describes the soil resistance to the  
194 lateral movement of a pile group through the use of a constant  $p$ -multiplier (i.e., the so-called group efficiency factor)



195 equally applied to all piles of the group (Adeel et al. 2020; Brown et al. 2001; Lemnitzer et al. 2010; Liu et al. 2020).  
 196 This group efficiency factor is commonly calculated as the average value of the  $p$ -multipliers for different pile rows,  
 197 and is adopted because different piles alternate the roles of leading and trailing piles during cyclic loading or earthquake  
 198 excitations. However, this constant reduction factor cannot correctly simulate the difference of lateral soil resistance  
 199 among different rows in a pile group under cyclic loading, leading to inaccurate estimates of differential curvatures for  
 200 piles in different rows.

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

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

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

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

$$p_{ud} = C_3 \cdot D \cdot \gamma \cdot h \quad (5)$$

210 where  $p$  is the lateral resistance of soil at the embedded depth  $h$ ;  $y$  denotes the lateral deflection of the pile at depth  $h$ ;  
 211  $p_u$  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  
 212  $p_{ud}$  denote the ultimate resistance of soil in the shallow and deep regions, respectively;  $n_h$  is the initial subgrade reaction  
 213 modulus of sand, which can be obtained from API specification as a function of the sand friction angle;  $\gamma$  is the soil  
 214 weight density;  $C_1$ ,  $C_2$ , and  $C_3$  are non-dimensional coefficients that depend on the effective friction angle (API 2007;  
 215 Chai and Song 2012).

216 The nonlinear asymmetric spring is approximatively modeled using the uniaxial material *QzSimple1* in *OpenSees*  
 217 (Boulanger et al. 1999), which has a behavior similar to the constitutive model for the  $p$ - $y$  spring in the compression

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

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

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

229 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  
 230 the soil resistance for the  $n$ -th row piles provided by the asymmetric spring; the superscript  $n = 1, 2, \dots, n_{max}$  denotes the  
 231 pile row number, and  $n_{max}$  is the total number of rows. As a result, the required input parameters of the *PySimple1*  
 232 material in *OpenSees*,  $p_{ult}$ , and  $y_{50}$  (Blanco et al. 2019), are given by:

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

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

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

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

$$z_{50} = y_{50} \quad (11)$$

236 where  $q_{ult}$  denotes the ultimate soil resistance provided by the asymmetric  $q$ - $z$  spring;  $z_{50}$  denotes the soil displacement  
 237 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  
 238 in the two opposite loading directions, the soil resistance corresponding to this pile row can be modeled more simply

239 by using only the symmetric  $p$ - $y$  springs with the appropriate value of the  $p$ -multiplier. For the numerical model of the  
 240 quasi-static test considered in this study, which involves a pile group foundation with three rows of piles in the loading  
 241 direction with a  $3D$  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  
 242  $f_m^{(2)} = f_{m,l}^{(2)} = f_{m,t}^{(2)} = 0.4$ .

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

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

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

245 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  
 246 lateral earth pressure at rest and is set equal to 0.4;  $\phi$  is the friction angle of sand;  $z_{50}$  is the displacement at which the  
 247 friction force reaches 50% of  $t_{ult}$ ;  $k$  denotes the initial tangent stiffness and can be expressed as a function of the friction  
 248 angle (Mosher 1984). Finally, the FE model of the benchmark example used for validation describes the boundary  
 249 conditions at the pile tips with vertical springs, the behavior of which is given by an ENT material with an initial  
 250 stiffness of  $1 \times 10^7$  kN/m, as shown in Figure 4h. The modeling parameter values for the quasi-static test used in Eqs.  
 251 (2) through (13) are given in Table 5.

## 252 **Numerical Model Validation**

253 The proposed FEM approach for pile group effect modeling under cyclic loading, referred to as proposed cyclic model  
 254 (PCM) hereinafter, is validated through a comparison with the experimental results available in Zhou et al. (2021a). In  
 255 order to assess the performance of this approach with those commonly used in the literature, two additional FE models  
 256 are built in *OpenSees*. The first additional FE model, referred to ordinary monotonic model (OMM), uses ordinary  
 257 symmetric  $p$ - $y$  springs with  $p$ -multipliers equal to 0.8, 0.4, and 0.3 for leading, middle, and trailing piles, respectively,  
 258 and is subjected to a monotonic pushover analysis with a maximum lateral displacement equal to 100 mm. These  $p$ -  
 259 multiplier values are equal to those recommended in AASHTO (2020) for a three-row pile group in sandy soil with a  
 260 spacing of  $3D$ . The second additional FE model, referred to as ordinary cyclic model (OCM), adopts a constant  $p$ -  
 261 multiplier (i.e., group efficiency factor) equal to 0.5 applied to all piles of the group, and is subjected to a quasi-static  
 262 cyclic loading. The value of the group efficiency factor is calculated as the average value of the  $p$ -multipliers for

263 different pile rows, as recommended by Brown et al. (2001). All other modeling details are identical for the three  
264 considered FE models. The lateral load is applied to the cap center using displacement-controlled loading and the non-  
265 linear residual equations of equilibrium are solved using the Newton-Raphson algorithm with the command *Newton* in  
266 *OpenSees* (Mazzoni et al. 2006). It is observed here that all three modeling methods for the pile group effect are based  
267 on the BNWF model, which is commonly used in practical applications. In addition, the computational effort associated  
268 to both PCM and OCM is almost identical; in fact, the clock time for both cyclic analyses were approximately 1280 s and  
269 1180 s, respectively, when using a personal computer with an Intel(R) Core (TM) i7-10750H CPU @ 2.60GHz and 32  
270 GB RAM.

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

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

291 2018) for different displacement levels, which is calculated as follows:

$$\varepsilon_{FE} = \frac{\sqrt{\frac{1}{N} \sum_{i=1}^N (\phi_{FE,i} - \phi_{exp,i})^2}}{\max_{1 \leq i \leq N} (|\phi_{exp,i}|)} \quad (14)$$

292 where  $\phi_{exp,i}$  and  $\phi_{FE,i}$  denote the experimentally-measured and numerically-predicted pile curvatures, respectively,  
293 at location  $i = 1, 2, \dots, N$  along the pile;  $N$  denotes the total number of the experimental curvature data points collected  
294 along a given pile; and  $FE = PCM, OCM, \text{ or } OMM$  indicates the considered FE model. It is observed that the predicted  
295 pile curvatures by the PCM and OMM are almost identical (i.e.,  $|\varepsilon_{PCM} - \varepsilon_{OMM}| \leq 2.6\%$  for all considered displacement  
296 levels and all pile rows), and present a good agreement with the experimental results (i.e.,  $2.1\% \leq \varepsilon_{PCM} \leq 10.2\%$  and  
297  $2.7\% \leq \varepsilon_{OMM} \leq 10.0\%$ ). When the pile group is subjected to cyclic loads, the PCM provides better predictions of the  
298 pile curvatures than the OCM, with  $\varepsilon_{PCM} < \varepsilon_{OCM}$ , except for leading piles at 10 mm displacement (for which  
299  $\varepsilon_{PCM} = 8.9\%$  and  $\varepsilon_{OCM} = 5.6\%$ ). In particular, the OCM underestimates the curvature in the belowground plastic hinge  
300 regions and overestimates the embedded depth of the plastic hinge for the leading piles, whereas it overestimates the  
301 curvature in the belowground plastic hinge regions and underestimate the embedded depth of the plastic hinge for the  
302 trailing piles.

### 303 **Parametric Sensitivity Analysis of Pile Group Foundations in Sandy Soils**

#### 304 *Parametric Study Matrix*

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

311 Eleven modeling parameters are investigated in this study: seven geometric parameters, i.e., pile configuration  
312 ( $P.C.$ ), pile length ( $L_p$ ), pile diameter ( $D$ ), pile center-to-center spacing ( $S$ ), scour depth ( $L_a$ ), longitudinal steel  
313 reinforcement ratio ( $\rho_l$ ), and transverse steel reinforcement ratio ( $\rho_s$ ); one loading parameter, i.e., axial load ratio of

314 the piles ( $\eta$ ); two material parameters, i.e., concrete strength ( $f_c$ ), and yield strength of the steel reinforcement ( $f_y$ ); and  
315 one soil parameter, i.e., relative density of sand ( $D_r$ ). A central composite design method is used to select the parameter  
316 combinations to be investigated, as shown in Table 7. For these combinations, C0 is selected as the central point, and  
317 every other combination is obtained by varying one parameter at a time, with three different values for each parameter.  
318 These parameter values are selected based on typical values encountered in practice and in the literature (Aviram et al.  
319 2008; Blanco et al. 2019; Das 2002; Jones et al. 2002). As a result, 23 cases are considered in this study.

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

### 327 ***Ductility Development and Performance Limit States of Scoured Pile Groups***

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

341 longitudinal steel reinforcement in any pile within the pile group, whichever occurs first; and (5) ultimate residual  
342 strength (URS), defined here as a 15% reduction of the lateral strength from its peak value (Ataei and Padgett 2012;  
343 Shen et al. 2021). A multiple-level displacement ductility index for a pile group is defined here as:

$$\mu_i = \frac{\Delta_i}{\Delta_1} \quad (15)$$

344 where  $\Delta_i$  represents the horizontal displacement of the cap center corresponding to any specific damage state  $i = 1, 2,$   
345 3, 4, and 5 (i.e., FAY, FBY, PLS, SSD, and URS limit states, respectively).

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

358 The pile length has negligible impact on the lateral resistance of the pile group and on the displacements  
359 corresponding to the different limit states. It is noted that this conclusion is valid here because the piles behave as deep  
360 foundations without uplift. As expected, the lateral strength of a pile group significantly increases for increasing pile  
361 diameters. More specifically, the peak lateral strength of the studied pile groups is equal to 1511 kN, 7527 kN, and  
362 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  
363 and the limit state displacements of a pile group. An increasing scour depth produces a reduction of the peak lateral  
364 strength of the pile groups and an increase of the displacement values corresponding to different limit states. In  
365 particular, the peak lateral strength of the pile groups are equal to 9988 kN, 7527 kN, and 5860 kN for a scour depth  
366 equal to  $3D$ ,  $5D$ , and  $7D$  (with  $D = 1.2$  m), respectively. Increasing the longitudinal steel reinforcement ratio can

367 enhance the lateral resistance of a pile group and increase the displacement corresponding to the first three limit states  
368 (FAY, FBY, and PLS). By contrast, the transverse steel reinforcement ratio has a negligible effect on the global lateral  
369 load-displacement response of the pile group before PLS, but it becomes significant in controlling the strength  
370 degradation after peak. An increasing axial load ratio slightly increases the peak lateral strength of a pile group, but  
371 also accelerates the crushing of the core concrete and triggers a rapid degradation of the lateral strength, which is an  
372 undesirable behavior for structures subjected to earthquake excitations. The increase of concrete strength can improve  
373 the lateral strength of a pile group, but it results in a more rapid degradation of the lateral strength. By contrast,  
374 increasing the yield strength of steel reinforcement can delay the lateral strength degradation after peak, but it is not  
375 the most desirable approach for improving the lateral resistance of a pile group. The relative density of sand has very  
376 small effects on the peak lateral strength and the limit state displacements of a pile group.

### 377 *Sensitivity Rankings of the Studied Parameters*

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

$$384 \quad sw_R = sw_R^{(+)} + sw_R^{(-)} = \frac{\Delta R^{(+)}}{R_0} + \frac{\Delta R^{(-)}}{R_0} = \left| \frac{R^{(+)} - R_0}{R_0} \right| + \left| \frac{R^{(-)} - R_0}{R_0} \right| \quad (16)$$

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

391 The global resistance of a pile group is quantified by three indexes, i.e.: (1) the lateral strength at the FAY limit



392 state, referred to as yield strength of a pile group hereinafter, beyond which the yielding of a pile group initiates; (2)  
 393 the strength enhancement coefficient of a pile group after yielding, denoted as  $SE$  and defined as follows:

$$SE = \frac{F_{PLS}}{F_{FAY}} \quad (17)$$

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

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

398 where  $F_{URS}$  denote the lateral strength at the URS limit state,  $\Delta_{FAY}$  and  $\Delta_{URS}$  denote the displacement at the FAY and  
 399 URS limit states, respectively.

400 Figure 8 shows the sensitivity rankings of the eleven parameters with respect to the global seismic resistance of the  
 401 studied pile groups. The parameters below the horizontal dashed line have a small effect on the studied response with a one-  
 402 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  
 403 (with a relative swing equal to 2.19), whereas strength enhancement coefficient and normalized strength degradation rate are  
 404 most sensitive to the axial load ratio (with a relative swing equal to 0.19 and 0.85, respectively). Increasing the pile diameter  
 405 can significantly improve the yield strength, raise the strength enhancement coefficient, and delay the strength degradation  
 406 of a pile group. Increasing the axial load ratio slightly enhance the yield strength of a pile group, but it considerably  
 407 accelerates the strength degradation and reduces the strength enhancement coefficient of a pile group, which is an undesirable  
 408 effect for the scoured pile groups. The scour is also undesirable for a pile group, because it significantly weakens the pile  
 409 group (e.g., the yield strength drops by 36% in this study when the scour depth increases from  $3D$  to  $7D$ ), decreases the  
 410 strength enhancement coefficient, and accelerates the strength degradation of a pile group. The transverse steel reinforcement  
 411 ratio has a small effect on the yield strength and the strength enhancement coefficient of a pile group, but it can significantly  
 412 delay the lateral strength degradation of a pile group after peak strength is reached (with a relative swing equal to 0.73). In  
 413 addition, the global resistance of a pile group is mostly insensitive to the relative density of sandy soil and to the pile length.  
 414 When compared with the yield strength and the normalized strength degradation rate, the strength enhancement coefficient  
 415 is relatively stable and shows less sensitivity to the studied parameters, because its maximum one-side relative swing is only

416 0.12.

417 Figure 9 presents the parameter sensitivity ranking for the residual displacement of a pile group at FBY limit state  
418 (denoted as  $DR_{FBY}$  hereinafter). The residual displacement is obtained by the following two analysis steps through the  
419 use of the PCM: (1) load the model to determine the lateral displacement of the pile group at FBY limit state, and (2)  
420 unload the model until it reaches a zero-lateral force state. The displacement corresponding to the zero-lateral force  
421 state is regarded as the residual displacement of the pile group (Zhou et al. 2021a). As shown in Figure9, the top five  
422 parameters to which the residual displacement of a pile group at the FBY limit state is sensitive are (in decreasing order  
423 of importance): (1) axial load ratio, (2) yield strength of the steel reinforcement, (3) longitudinal steel reinforcement  
424 ratio, (4) pile diameter, and (5) scour depth in sequence, corresponding to the relative swings of 0.91, 0.54, 0.51, 0.49,  
425 and 0.26, respectively. Their increase leads to the increase of the residual displacement of a pile group, which is  
426 detrimental for the post-earthquake recovery efforts. The pile length and transverse steel reinforcement ratio affect less  
427 the residual displacement of a pile group corresponding to the FBY state since the one-side relative swing is less than  
428 0.05.

429 Figure 10 shows the sensitivity rankings of the different parameters with respect to the displacement ductility of  
430 a pile group at the FBY, PLS, and SSD limit states. The top four parameters to which the FBY displacement ductility  
431 of a pile group is sensitive are (in decreasing order of importance)  $P.C.$ , scour depth, pile center to center spacing, and  
432 pile diameter. The other remaining seven parameters have a small effect on the FBY displacement ductility since the  
433 one-side relative swing is less than 0.05. The  $2 \times 3$  and  $3 \times 3$  pile groups present an almost identical FBY displacement  
434 ductility since they have the same pile group effect. The axial load ratio affects the most the displacement ductility of  
435 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  
436 small impact on the ductility of a pile group at the FBY state (with a relative swing smaller than 0.006). The higher the  
437 axial load ratio causes a lower pile group ductility corresponding to PLS and SSD states. The increase of the scour  
438 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  
439 pile group at the SSD state. Increasing the transverse steel reinforcement ratio improves the ductility of a pile group at  
440 the SSD limit state; however, the transverse steel reinforcement ratio has a negligible impact on the ductility of the pile  
441 group at the FBY and PLS limit states. The displacement ductility of a pile group is insensitive to the pile length and  
442 the relative density of sand.

## 443 *Considerations for Seismic Performance Improvements*

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

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

## 466 **Conclusions**

467 This study investigates the seismic performance of scoured pile group foundations. A new practical finite element  
468 modeling approach for the pile group effect is first proposed and validated by the experimental data available in the  
469 literature. The proposed approach can simulate the soil resistance difference among different pile rows in a pile group

470 under lateral cyclic loads and provides more accurate local pile curvature results than the currently adopted  
471 methodology based on a constant pile group factor. It is also observed that both approaches provide almost identical  
472 results in terms of global response quantities, e.g., global lateral force-displacement response, which are in good  
473 agreement with existing experimental results. By using the validated model, an in-depth parametric analysis is  
474 performed to explore parameter sensitivity with respect to the seismic performance of a pile group foundation. The  
475 main findings of this parameter sensitivity analysis are as follows:

- 476 (1) Scour significantly weakens the seismic capacity of a pile group. It reduces the lateral strength and displacement  
477 ductility of a pile group, and increases the residual displacement of a pile group at the first belowground yielding  
478 (FBY) limit state (corresponding to an easy-to-repair condition).
- 479 (2) Increasing the pile diameter is the most effective measure to improve the seismic performance of a scoured pile  
480 group. Increasing the concrete strength, the yield strength of the steel reinforcement, and the longitudinal steel  
481 reinforcement ratio can increase the yield strength of a pile group. However, increasing the yield strength of the  
482 steel reinforcement and the longitudinal steel reinforcement ratio can also increase the residual displacement of a  
483 pile group at the FBY state. Higher axial load ratio slightly increases the lateral strength of the pile group foundation,  
484 but also significantly increases its residual displacement, reduces its lateral strength enhancement coefficient, and  
485 accelerates its lateral strength degradation. The transverse reinforcement ratio has negligible effects on the yield  
486 strength and the residual displacement at the FBY limit state of a pile group; however, a higher transverse  
487 reinforcement ratio can decrease the degradation rate of the lateral strength. For a deep foundation in medium dense  
488 sand, the pile length (for values higher than or equal to 30 times the pile diameter), pile spacing (for values smaller  
489 than or equal to five times the pile diameter), and relative density of sand (for values between 40% and 60%) have  
490 negligible effects on seismic performance of a scoured pile group foundation.

491 This study uses a set of quasi-static tests available in the literature to validate the proposed pile group effect  
492 modeling approach for pile group foundations subjected to lateral cyclic loads. It is recommended to validate this  
493 modeling approach also for pile group foundations subjected to dynamic conditions, e.g., seismic shakings. This study  
494 mainly focuses on scoured pile groups in a homogeneous sand subjected to lateral loads. Further studies are needed to  
495 quantify the impacts of different soil conditions and profiles. In addition, future numerical studies should investigate  
496 the use of incremental dynamic analysis to develop fragility curves and achieve a better understanding of the ductile  
497 behavior and the failure mechanisms of bridges with scoured pile group foundations. It is also suggested to investigate

498 whether the  $p$ -multipliers vary with the depth and with the lateral deformation level of the pile group foundation, as  
499 well as how this potential variation affects the performance of a scoured pile group foundation.

## 500 **Data Availability Statement**

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

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## 513 **References**

- 514 AASHTO (American Association of State Highway and Transportation Officials). (2020). *AASHTO LRFD bridge design*  
515 *specifications*. Washington, D.C., Washington, D.C.
- 516 AASHTO (American Association of State Highway and Transportation Officials). (2022). *AASHTO guide specifications for*  
517 *LRFD seismic bridge design*. Washington, D.C.
- 518 Abbas, S. A., Rekabi, W. S. Al, and Aboodi, A. H. Al. (2016). "Analysis of laterally loaded (2×2) square pile groups using finite  
519 element method." *Sixth International Conference On Advances in Civil and Structural Engineering - CSE 2016*, Institute  
520 of Research Engineers and Doctors, Kuala Lumpur, Malaysia, 7–12.
- 521 Adeel, M. B., Jan, M. A., Aaqib, M., and Park, D. (2020). "Development of simulation based  $p$ -multipliers for laterally loaded  
522 pile groups in granular soil using 3d nonlinear finite element model." *Applied Sciences*, 11(1), 26.
- 523 Albusoda, B. S., Al-Saadi, A. F., and Jasim, A. F. (2018). "An experimental study and numerical modeling of laterally loaded  
524 regular and finned pile foundations in sandy soils." *Computers and Geotechnics*, 102, 102–110.
- 525 Alipour, A., and Shafei, B. (2016). "Seismic resilience of transportation networks with deteriorating components." *Journal of*

526        *Structural Engineering*, 142(8), C4015015.

527 Allotey, N., and El Naggar, M. H. (2008). "A numerical study into lateral cyclic nonlinear soil–pile response." *Canadian*  
528        *Geotechnical Journal*, 45(9), 1268–1281.

529 API (American Petroleum Institute). (2007). *Recommended practice for planning, designing and constructing fixed offshore*  
530        *platforms - working stress design. RP 2A-WSD, Washington, DC.*

531 Ataei, N., and Padgett, J. E. (2012). "Fragility assessment of coastal bridges under hurricane events using enhanced probabilistic  
532        capacity models." *Advances in Hurricane Engineering*, American Society of Civil Engineers, Reston, VA, 691–702.

533 Aviram, A., Mackie, K., and Stojadinovic, B. (2008). *Guidelines of nonlinear analysis of bridge structures in California.*  
534        *Berkeley, CA: Pacific Earthquake Research Center, Univ. of California, Berkeley, PEER 2008/03.*

535 Banerjee, S., Stanton, J. F., and Hawkins, N. M. (1987). "Seismic performance of precast prestressed concrete piles." *Journal of*  
536        *Structural Engineering*, 113(2), 381–396.

537 Barbato, M., Gu, Q., and Conte, J. P. (2010). "Probabilistic push-over analysis of structural and soil-structure systems." *Journal*  
538        *of Structural Engineering*, 136(11), 1330–1341.

539 Blanco, G., Ye, A., Wang, X., and Goicolea, J. M. (2019). "Parametric pushover analysis on elevated RC pile-cap foundations  
540        for bridges in cohesionless soils." *Journal of Bridge Engineering*, 24(1), 04018104.

541 Boulanger, R. W., Curras, C. J., Kutter, B. L., Wilson, D. W., and Abghari, A. (1999). "Seismic soil-pile-structure interaction  
542        experiments and analyses." *Journal of Geotechnical and Geoenvironmental Engineering*, 125(9), 750–759.

543 Boulanger, R. W., Kutter, B. L., Brandenburg, S. J., Singh, P., and Chang, D. (2003). *Pile foundations in liquefied and laterally*  
544        *spreading ground during earthquakes: centrifuge experiments & analyses.* Sacramento, CA, Davis: University of  
545        California.

546 Brandenburg, S. J., Boulanger, R. W., Kutter, B. L., and Chang, D. (2007). "Static pushover analyses of pile groups in liquefied  
547        and laterally spreading ground in centrifuge tests." *Journal of Geotechnical and Geoenvironmental Engineering*, 133(9),  
548        1055–1066.

549 Brown, D. A., Morrison, C., and Reese, L. C. (1988). "Lateral load behavior of pile group in sand." *Journal of Geotechnical*  
550        *Engineering*, 114(11), 1261–1276.

551 Brown, D. A., O'Neill, M. W., Hoit, M., McVay, M., Nagger, M. H. El, and Chakraborty, S. (2001). *Static and dynamic lateral*  
552        *loading of pile groups.* NCHRP Report No. 461, Transportation Research Board, National Research Council, Washington,  
553        D.C.

554 Capers, H. A., Huang, C., Kulicki, J. M., Murphy, T., Lee, G. C., Liang, Z., Shen, J. J., Shinozaka, M., and Yen, W. P. (2013).  
555        *Principles of multiple-hazard design for highway bridges.* Federal Highway Administration, Washington, DC.

556 Chai, Y. H., and Hutchinson, T. C. (2002). "Flexural strength and ductility of extended pile-shafts II: experimental study." *Journal of Structural Engineering*, 128(5), 595–602.

557        *Journal of Structural Engineering*, 128(5), 595–602.

558 Chai, Y. H., and Song, S.-T. (2012). "Inelastic deformation of extended pile-shafts." *Journal of Engineering Mechanics*, 138(7),

559 842–852.

560 Christensen, D. S. (2006). “Full scale static lateral load test of a 9 pile Group in sand.” Brigham Young University, Provo, Utah.

561 Das, B. M. (2002). *Principles of foundation engineering. McGraw-Hill handbooks*, Washington DC.

562 Fayyazi, M. S., Taiebat, M., Finn, W. D. L., and Ventura, C. E. (2012). “Evaluation of group factor method for analysis of pile

563 groups.” *15th World Conference on Earthquake Engineering (15WCEE)*, (June 2014).

564 Filippou, F. C., Popov, E. P., and Bertero, V. V. (1983). *Effects of bond deterioration on hysteretic behaviour of reinforced*

565 *concrete joints. Berkeley, CA: Earthquake Engineering Research Center, Univ. of California, Berkeley, EERC-83/19*.

566 Heidari, M., El Naggar, H., Jahanandish, M., and Ghahramani, A. (2014). “Generalized cyclic p–y curve modeling for analysis of

567 laterally loaded piles.” *Soil Dynamics and Earthquake Engineering*, 63, 138–149.

568 Hussien, M. N., Tobita, T., Iai, S., and Karray, M. (2016). “Soil-pile-structure kinematic and inertial interaction observed in

569 geotechnical centrifuge experiments.” *Soil Dynamics and Earthquake Engineering*, 89, 75–84.

570 Hutchinson, T. C., Chai, Y. H., and Boulanger, R. W. (2005). “Simulation of full-scale cyclic lateral load tests on piles.” *Journal*

571 *of Geotechnical and Geoenvironmental Engineering*, 131(9), 1172–1175.

572 Hutchinson, T. C., Chai, Y. H., Boulanger, R. W., and Idriss, I. M. (2004). “Inelastic seismic response of extended pile-shaft-

573 supported bridge structures.” *Earthquake Spectra*, 20(4), 1057–1080.

574 Jones, A. L., Kramer, S. L., and Arduino, P. (2002). *Estimation of uncertainty in geotechnical properties for performance-based*

575 *earthquake*. Pacific Earthquake Engineering Research Center, PEER 2002/16, Berkeley, CA.

576 Kawashima, K., Takahashi, Y., Ge, H., Wu, Z., and Zhang, J. (2009). “Reconnaissance report on damage of bridges in 2008

577 Wenchuan, China, earthquake.” *Journal of Earthquake Engineering*, 13(7), 965–996.

578 Kim, B. T., and Yoon, G. L. (2011). “Laboratory modeling of laterally loaded pile groups in sand.” *KSCE Journal of Civil*

579 *Engineering*, 15(1), 65–75.

580 Kotthuas, M. (1992). “Zum tragverhalten von horizontal belasteten pfahl- reihen aus langen pfählen in sand.” (In German), PhD

581 dissertation, Schriftenreihe des Instituts für Grundbau, Ruhr-Univ., Bochum, Ger- many, Heft 18.

582 Kramer, S. L., Arduino, P., and HyngSuk, S. (2008). *Using Opensees for Performance-Based evaluation of bridges on*

583 *liquefiable soils. Pacific Earthquake Engineering Research Center*.

584 Lemnitzer, A., Khalili-Tehrani, P., Ahlberg, E. R., Rha, C., Taciroglu, E., Wallace, J. W., and Stewart, J. P. (2010). “Nonlinear

585 efficiency of bored pile group under lateral loading.” *Journal of Geotechnical and Geoenvironmental Engineering*,

586 136(12), 1673–1685.

587 Liu, T., Wang, X., and Ye, A. (2020). “Roles of pile group and cap-rotation effects on seismic failure mechanisms of partially-

588 embedded bridge foundations: Quasi-static tests.” *Soil Dynamics and Earthquake Engineering*, 132, 106074.

589 Mander, J. B., Dutta, A., and Goel, P. (1998). *Capacity design of bridge piers and the analysis of overstrength*. MCEER-98-

590 0003. Buffalo, NY: Dept. of Civil, Structural and Environmental Engineering, Univ. at Buffalo.

591 Mander, J. B., Priestley, M. J. N., and Park, R. (1988). “Theoretical stress-strain model for confined concrete.” *Journal of*

592           *Structural Engineering*, 114(8), 1804–1826.

593 Matlock, H., and Ripperger, E. A. (1956). “Procedures and instrumentation for tests on a laterally loaded pile.” *Eighth Texas*  
594           *Conference on Soil Mechanics and Foundation Engineering*, Special Publication No. 29, Bureau of Engineering Research,  
595           University of Texas.

596 Mazzoni, S., McKenna, F., Scott, M. H., and Fenves, G. L. (2006). *OpenSees command language manual*. Pacific Earthquake  
597           *Engineering Research Center*.

598 McKenna, F. (2011). “OpenSees: A framework for earthquake engineering simulation.” *Computing in Science and Engineering*,  
599           13(4), 58–66.

600 McVay, M., Casper, R., and Shang, T. I. (1995). “Lateral response of three-row groups in loose to dense sands at 3D and 5D pile  
601           spacing.” *Journal of Geotechnical Engineering*, 121(5), 436–441.

602 McVay, M., Zhang, L., Molnit, T., and Lai, P. (1998). “Centrifuge testing of large laterally loaded pile groups in sands.” *Journal*  
603           *of Geotechnical and Geoenvironmental Engineering*, 124(10), 1016–1026.

604 Mosher, R. L. (1984). *Load-transfer criteria for numerical analysis of axially loaded piles in sand. Part 1 Load-transfer criteria*.  
605           U. S. Army Engineer Waterways Experiment Station. Vicksburg, Miss.

606 Park, R., and Falconer, T. J. (1983). “Ductility of prestressed concrete piles subjected to simulated seismic loading.” *PCI*  
607           *Journal*, 28(5), 112–144.

608 Reese, L. C., Wang, S. T., and Vasquez, L. (2006). “Computer program GROUP Version 7.0—Technical manual.” Ensoft Inc,  
609           Austin, Tex.

610 Rizzo, F., Barbato, M., and Sepe, V. (2018). “Peak factor statistics of wind effects for hyperbolic paraboloid roofs.” *Engineering*  
611           *Structures*, 173, 313–330.

612 Rollins, K. M., Lane, J. D., and Gerber, T. M. (2005). “Measured and computed lateral response of a pile group in sand.” *Journal*  
613           *of Geotechnical and Geoenvironmental Engineering*, 131(1), 103–114.

614 Scott, B. D., Park, R., and Priestley, M. J. N. (1982). “Stress-strain behavior of concrete confined by overlapping hoops at low  
615           and high strain rates.” *ACI Journal Proceedings*, 79(1), 13–27.

616 Shang, Y., Alipour, A., and Ye, A. (2018). “Selection of input motion for seismic analysis of scoured pile-supported bridge with  
617           simplified models.” *Journal of Structural Engineering*, 144(8), 04018099.

618 Shen, Y., Liu, X., Li, Y., and Li, J. (2021). “Cyclic tests of precast post-tensioned concrete filled steel tubular (PCFT) columns  
619           with internal energy-dissipating bars.” *Engineering Structures*, 229, 111651.

620 Song, S.-T., Hu, T.-F., and Chiou, D.-J. (2020). “Influence of riverbed scour on the performance of bridges subjected to lateral  
621           seismic loads.” *Journal of Earthquake Engineering*, 00(00), 1–32.

622 Vakili, A., Zomorodian, S. M. A., and Totonchi, A. (2021). “Laboratory and three-dimensional numerical modeling of laterally  
623           loaded pile groups in sandy soils.” *Iranian Journal of Science and Technology, Transactions of Civil Engineering*, Springer  
624           International Publishing, 45(4), 2623–2636.



625 Wang, S.-C., Liu, K.-Y., Chen, C.-H., and Chang, K.-C. (2015). "Experimental investigation on seismic behavior of scoured  
626 bridge pier with pile foundation." *Earthquake Engineering & Structural Dynamics*, 44(6), 849–864.

627 Wang, S., Kutter, B. L., Chacko, M. J., Wilson, D. W., Boulanger, R. W., and Abghari, A. (1998). "Nonlinear seismic soil-Pile  
628 structure interaction." *Earthquake Spectra*, Earthquake Engineering Research Institute, 14(2), 377–396.

629 Wang, X., Ye, A., He, Z., and Shang, Y. (2016). "Quasi-static cyclic testing of elevated rc pile-cap foundation for bridge  
630 structures." *Journal of Bridge Engineering*, 21(2), 04015042.

631 Wang, X., Ye, A., Shang, Y., and Zhou, L. (2019). "Shake-table investigation of scoured RC pile-group-supported bridges in  
632 liquefiable and nonliquefiable soils." *Earthquake Engineering & Structural Dynamics*, 48(11), 1217–1237.

633 Wang, Z., Dueñas-Osorio, L., and Padgett, J. E. (2014). "Influence of scour effects on the seismic response of reinforced  
634 concrete bridges." *Engineering Structures*, 76, 202–214.

635 Wardhana, K., and Hadipriono, F. C. (2003). "Analysis of recent bridge failures in the united states." *Journal of Performance of*  
636 *Constructed Facilities*, 17(3), 144–150.

637 Wei, X., Wang, Q., and Wang, J. (2008). "Damage patterns and failure mechanisms of bridge pile foundation under earthquake." *14th World Conference on Earthquake Engineering*, Beijing, China, 1–8.

638

639 Zhang, J., and Hutchinson, T. C. (2012). "Inelastic pile behavior with and without liquefaction effects." *Soil Dynamics and*  
640 *Earthquake Engineering*, Elsevier, 36, 12–19.

641 Zhou, L., Barbato, M., and Ye, A. (2021a). "Experimental investigation of postearthquake vertical load-carrying capacity of  
642 scoured reinforced concrete pile group bridge foundations." *Journal of Bridge Engineering*, 26(12), 1–14.

643 Zhou, L., Shahria Alam, M., Song, A., and Ye, A. (2022a). "Probability-based residual displacement estimation of unbonded  
644 laminated rubber bearing supported highway bridges retrofitted with Transverse Steel Damper." *Engineering Structures*,  
645 Elsevier Ltd, 272, 115053.

646 Zhou, L., Ye, A., and Chen, F. (2021b). "Research on stress-strain constitutive model of concrete confined by galvanized iron  
647 wires." (*In Chinese*), *China Civil Engineering Journal*, 54(8), 111–119.

648 Zhou, L., Ye, A., and Chen, F. (2022b). "Postearthquake vertical load-carrying capacity of extended pile shafts in cohesionless  
649 soils: quasi-static test and parametric studies." *Journal of Bridge Engineering*, 27(8), 1–16.

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

Parameter	Unit	Value
Unit weight ( $\gamma$ )	kN/m <sup>3</sup>	15.95
Moisture content ( $w$ )	%	0.16
Average relative density ( $D_r$ )	%	55
Friction angle ( $\phi$ )	Degree	33

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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

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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
$\phi$ 6mm rebars	216,353	429	670	0.120
$\phi$ 3.5mm GIW	135,441	317	421	0.148

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658 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)	$S/D$	1 <sup>st</sup> row	2 <sup>nd</sup> row	3 <sup>rd</sup> 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 ( $\phi = 35\text{--}40^\circ$ ) 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 ( $D_r = 33\%$ ) Medium dense sand ( $D_r = 55\%$ ) Medium loose sand ( $D_r = 33\%$ ) Medium dense sand ( $D_r = 55\%$ )	1/45 scale centrifuge test	3×3	0.0095	3	0.65	0.45	0.35	
				3×3	0.0095	3	0.8	0.45	0.3	
				3×3	0.0095	5	1	0.85	0.7	
				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 ( $D_r = 73\%$ ) Medium dense sand ( $D_r = 55\%$ )	Small-scale tests	1×3	0.012	4	0.85	0.6	0.45	
				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

659 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
$\gamma$	kN/m <sup>3</sup>	15.95
$n_h$	kN/m <sup>3</sup>	27145.0
$k$	kN/m <sup>3</sup>	22620.0

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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%

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Table 7. Parameter matrix and studied cases

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

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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

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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 $\mu_{FBy}$
$P.C.$	Pile configuration	++	—	—	—	— — — —
$L_p$	Pile length	\	\	\	\	\
$D$	Pile diameter	+++	+	+	×	×
$S$	Pile center to center spacing	—	\	+	+	—
$L_a$	Scour depth	×	\	×	×	××
$\rho_l$	Longitudinal steel reinforcement ratio	+	+	×	×	\
$\rho_s$	Transverse reinforcement ratio	\	\	++	\	\
$\eta$	Axial load ratio	+	×××	×××	×××	\
$f_c$	Concrete strength	+	××	×	—	\
$f_y$	Yield strength of reinforcement	+	\	\	××	\
$D_r$	Relative density of sand soil	\	\	\	+	\

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

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

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

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

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

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

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

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

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

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