Roles of pile-group and cap-rotation effects on seismic failure mechanisms of partially-embedded bridge foundations: Quasi-static tests

Tengfei Liu a, Xiaowei Wang b, Aijun Ye a,∗

a State Key Laboratory of Disaster Reduction in Civil Engineering, Tongji University, No. 1239 Siping Road, Shanghai, 200092, China
b College of Civil and Transportation Engineering, Hohai University, No. 1 Xikang Road, Nanjing, 210024, China

ARTICLE INFO

Keywords:
RC pile-group foundations
Partially-embedded
Scour
Seismic failure mechanism
Quasi-static test
Cap rotation
Axial load variations
Pile group effects

ABSTRACT

Pile foundations, especially partially embedded in soils, are vulnerable parts of bridges under seismic loadings. In this study, to investigate seismic failure mechanisms of partially-embedded reinforced concrete (RC) pile-group foundations, a series of quasi-static cyclic loading tests were conducted on 1 × 1, 2 × 2 and 2 × 3 pile-foundation specimens in sand. Observed hysteretic behavior and pile damage were recorded, along with pile curvature distributions. Then, numerical models based on the Beam-on-Nonlinear-Winkler-Foundation (BNWF) approach were developed and validated by test data. The failure process of each specimen was reached, characterized by the sequence and locations of plastic hinges on piles. Combining the test and numerical results, seismic failure mechanisms of all partially-embedded pile specimens with different layouts are revealed: side piles yield before center piles, each pile forms two plastic hinges, and pile heads yield before underground portions. Special attentions are paid to the roles of cap rotation and pile-group effect (PGE) on the failure mechanisms. The results reveal that the existence of PGE leads to lower lateral loads and greater displacements at different limit states such as the first yielding and ultimate states etc., while the impact of cap rotation only increases corresponding lateral displacements at these limit states.

1. Introduction

Reinforced concrete (RC) pile foundations are extensively utilized in bridges because of economical and easy-to-build advantages. Typically, pile foundations can be categorized into two types: (1) fully embedded and (2) partially embedded pile foundations. A fully embedded pile foundation can sometimes become partially embedded due to natural hazards, such as riverbed scour [1–4]. According to previous studies [5, 6], about 60% bridge failures in the U.S. during the past 30 years were related to the scour of bridge foundations. Compared with fully embedded pile foundations, partially embedded ones have unsupported portions, which may reduce the stability and lateral resistance of pile foundations. In this regard, assessing seismic behavior of partially-embedded pile foundations draws increasing attention.

Currently, bridge design guidelines [7–9] adopt the capacity design strategy, in which inelastic behavior on pile foundations were not allowed. To reach this, a large plenty of reinforcements are needed for the piles, inevitably reducing the cost-effectiveness. Nevertheless, recent post-earthquake reconnaissance [10–12] still reported plastic damage to pile foundations. In this regard, utilizing pile foundations as energy-dissipating components tends to be a reasonable choice. To this end, it is necessary to investigate seismic failure mechanisms of pile foundations.

Quasi-static testing is an effective method to investigate the seismic behavior of pile foundations. In pioneering work [13–16], single-pile specimens were adopted and subjected to predefined loading patterns, which may not accurately represent the complicated soil-pile-interaction problem. To avoid this limitation, Chai and Hutchinson [17] conducted field tests on full-scale extended pile-shafts (i.e., single pile-column) in homogenous cohesionless soil to investigate flexural strengths and ductile capacities of the test piles. Compared with single piles, pile-group foundations have two instinct characteristics. One is the pile-group effect (PGE) induced by limited pile-to-pile distances as well as the complex soil-pile-interaction [18,19], the other is cap-rotation effect induced by the rotation of the cap in a pile group under lateral cyclic loads, which inevitably lead to variations of axial loads on piles. These variations of axial loads probably influence yielding and ultimate curvature capacities as well as curvature demands of pile sections [20]. Regarding this, the seismic failure mechanisms of pile-group foundations under lateral loadings are expected to be
different from single piles. Very few studies have been reported in this field. Ye et al. [21] studied the seismic performance of $2 \times 2$ elevated pile-cap foundations (EPFs, can be regarded as partially-embedded pile-group foundations) in sands by quasi-static tests. Wang et al. [20] extended this work and revealed seismic failure mechanisms of $2 \times 3$ EPFs with varying aboveground heights. However, the combined/separate impact(s) of cap rotation and PGE were not documented in these studies.

On the other hand, various numerical and analytical methods have been developed to predict lateral responses of soil-pile systems [22–30]. Generally, the BNWF method [31–33] is more advocated in practice due to its merits of low computational costs and easy applications, as compared to the three-dimensional finite element (FE) method with soil continua [34–38].

With this in view, the scope of this paper is to investigate seismic failure mechanisms of partially-embedded pile-group foundations for bridges, emphasizing the roles of cap rotation and PGE. First, quasi-static loading tests were carried out on three specimens with different pile layouts ($1 \times 1$, $2 \times 2$ and $2 \times 3$). Global hysteretic behavior, pile physical damage, and curvature distributions along piles were interpreted. Then based on the BNWF approach, numerical models are built in OpenSees [39] and verified using the experimental results. After that, seismic failure mechanisms of partially-embedded RC pile-group foundations with different pile layouts are revealed through comprehensive analyses on the experimental and numerical results, among which the combined/separate impact(s) of cap rotation and PGE are highlighted.

2. Experimental program

2.1. Test setup

The prototype structure in this study is a $2 \times 3$-pile-group-supported river-crossing simply supported multi-span girder bridge with individual spans of 30 m and deck-width of 16.25 m. The $2 \times 3$ pile-group prototype, with 9.0-m diameter and 27.0-m length, is cast-in-place bored and partially embedded into sandy deposits. The embedded depth of piles is approximately 22.2 m, corresponding to an aboveground height of 4.8 m.

Scaled pile specimens were prepared for quasi-static tests. Before introducing the similarity coefficients, it is worth noting that for laboratory tests of soil-structure specimens with limited dimension of soil box ($3.2 \times 1.6 \times 4.2$ m ($\text{Length} \times \text{Width} \times \text{Height}$)) under 1g gravitational field, it is very difficult to perfectly follow similarity laws (e.g., Iai [40]) in terms of both the soil and pile, especially for RC piles in soils [19]. Therefore, only the piles are scaled down, while soils are prepared to have reasonable relative densities that can provide appropriate boundary conditions to the models.

The similarity coefficients in length and force of the studied specimens are 1/6 and 1/36, respectively. In addition, different specimens were prepared based on the scaled $2 \times 3$ pile groups by varying the pile layouts to $1 \times 1$ and $2 \times 2$. In total, three 1/6-scale RC pile specimens, characterized by different layouts ($1 \times 1$, $2 \times 2$ and $2 \times 3$), were tested to investigate seismic failure mechanisms. The units for test setup descriptions and associated results hereinafter are all on the scaled models. For construction expedience, square section is used. The width of pile section is $D = 0.15$ m. The specimens are labeled as S1, S4, and S6, respectively. Table 1 summarizes the configuration of test models. Note that differences in cap dimensions between the studied cases should have little effect on the lateral behavior of pile foundations [41]. Fig. 1 illustrates the test setup and pile layouts for all specimens. Each specimen was embedded in the same soil box of $3.2 \times 1.6 \times 4.2$ m ($\text{Length} \times \text{Width} \times \text{Height}$). The boundary condition effect of this soil box has been examined and verified by Wang et al. [20] and will be re-verified in this study. The pile center-to-center spacing is 3D, the aboveground height of piles is $5.33D = 0.8$ m and the embedded depth is long enough ($24.67D = 3.7$ m) to develop full underground pile damage [42,43]. In this study, constant vertical loads were applied on caps to simulate initial vertical loads provided by superstructures, corresponding to a nominal axial compressive ratio $\alpha = P/\alpha \gamma_0 = 0.05$, for each pile, where $P = \text{vertical load on each pile} = 47.44$ kN, $\gamma = \text{uniaxial compressive strength of concrete cover}$, $\gamma_0 = \text{gross area of piles}$. It is worth noting that this axial compressive ratio, $\alpha$, (i.e., 0.05) falls into the common range for practice based on engineering judgements of the authors.

For the single pile case S1 shown in Fig. 1(a), both the joints A and B of the horizontal actuator were locked against rotation while the joint C of the vertical actuator was free to rotate. In this regard, the small pile-cap of the single pile S1 is not free to rotate and its rotation is partially restrained by the horizontal actuator. In other words, the head of the single pile S1 is not free, thereby it can be approximately regarded as an individual pile among a pile-group, but with neither pile-group effect nor cap-rotation effect (note that in this paper, cap-rotation effect refers to the variation of pile axial loads induced by cap rotation of pile-group, here in the case S1 for single pile, constant axial load was imposed). It should be noted that the restraint condition of single pile case may be not completely the same to the pile-group cases, but the setup of the actuators is what we can do the best at hand to achieve a similar restraint condition, as close as possible, among them. Note that the close restraint condition among the studied cases will be further justified later in Section 3.4 using curvature distribution responses.

The soil profile consists of two layers: (1) bottom layer: compacted gravel (0.5 m thick) and (2) upper layer: homogenous dense sand (3.5 m thick). Note that the gravel layer in S1 could resistant vertical load satisfactorily as expected. However, in pile-group cases (S4 and S6), to ensure the lateral loadings run successfully during the tests, steel blocks were placed at the bottom of the soil box (Fig. 1(b and c)) to avoid specimen settlements induced by the excessive axial loads for the gravel layer. The seismic behavior of pile foundations under lateral loading is affected by the vertical loads on piles. As mentioned above, the axial loads on piles are designed based on the compressive capacity of pile section, rather than the vertical resistance offered by soils and pile tips. Though S1, S4, and S6 are not strictly the same in pile-tip support, the designed vertical loads on each pile are equal (axial compressive ratio $= 0.05$) and remain stable during tests. Considering this, though there were tiny settlement of S1 and negligible vertical displacements of S4/S6 during tests, the seismic behavior is hardly affected by the difference of pile-tip boundary conditions between S1 and S4/S6. So the pile tips of all specimens can be modeled as vertically fixed in numerical models.

### Table 1

<table>
<thead>
<tr>
<th>Case</th>
<th>Pile layout</th>
<th>RC cap</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Section width (m)</td>
<td>Full length (m)</td>
</tr>
<tr>
<td>S1</td>
<td>1 x 1</td>
<td>0.15</td>
</tr>
<tr>
<td>S4</td>
<td>2 x 2</td>
<td>0.15</td>
</tr>
<tr>
<td>S6</td>
<td>2 x 3</td>
<td>0.15</td>
</tr>
</tbody>
</table>

Fig. 2 shows the reinforcing configuration of individual piles. The longitudinal steel ratio was 0.02, provided by four $\Phi$12 rebars, with 2 cm concrete cover. The pile was confined by $\Phi$6 transverse hooped bars at vertical intervals of 5 cm in the region beneath the cap up to 200 cm.
In the remaining portion (Part B in Fig. 2(a)), the same transverse bars were utilized but at larger vertical intervals of 10 cm. The volumetric transverse reinforcement ratios in Part A and B were 0.036 and 0.018, respectively. Additionally, to ensure rigid pile-cap connections, the rebars were extended into the RC cap with sufficient lengths (30 cm, 30 cm, and 50 cm for S1, S4, and S6, respectively) [20].

The uniaxial compressive strength of concrete was $f_c = 42.14$ MPa. According to tension tests of the longitudinal rebars, a well-defined yield stress of $f_y = 542.44$ MPa was obtained, together with an elastic modulus of 206 GPa. The ultimate stress was $f_u = 695.10$ MPa, corresponding to the ultimate strain of $\varepsilon_{u,s} = 0.099$. By contrast, the transverse bar did not have a well-defined yield stress. The nominal yield stress corresponding to 0.2% strain was $f_{y}' = 320.13$ MPa.

The software XTRACT [44] was utilized to ascertain the moment-curvature relationship of the pile section (Fig. 3). Note that the first-yielding curvature ($\phi_y$) corresponds to the first yielding of the longitudinal bar, whereas the ultimate curvature ($\phi_u$) is determined by the crushing of the concrete core or snapping of the longitudinal bar, whichever occurs first. The ultimate strain of concrete core ($\varepsilon_{u}$) is given by Eq. (1) [45].

$$\varepsilon_{u} = 0.004 + \frac{0.9f_{y}'}{300}$$

where $\rho_s$ = transverse volumetric ratio of pile, $f_{y}'$ = yield stress of transverse bar (MPa).

In this study, the equivalent-yielding state of the specimen is defined as that the equivalent-yielding curvature of the pile-section ($\phi_e$) is firstly reached. Here $\phi_e$ is determined by the ideally bilinear moment-curvature relationship considering energy equivalence. As indicatively depicted in Fig. 3, for specimen S1 with the pre-loaded vertical load of 47.44 kN, $\phi_e = 0.0354$ rad/m and $\phi_u = 1.999$ rad/m.

### 2.3. Soil details

This test employed yellow silicon sand to prepare the soil deposit, which is classified as poorly graded sand (SP) based on the Unified Soil Classification System [46]. The grading curve of test sand is shown in Fig. 4. The maximum and minimum dry bulk densities were 16.68 kN/m$^3$ and 14.03 kN/m$^3$, respectively. In each case, the measured unit weight was approximately 15.62 kN/m$^3$, corresponding to a relative density of $D_r = 65\%$. The internal friction angle was 31°. Main physical properties of test sand are summarized in Table 2.

As mentioned above, the gravel layer only serves in increasing vertical resistances of piles in the tests. Please note, due to the limitation of lab facilities, the gravel layer underlying the homogenous sand could not
prepared layer-by-layer, each in approximate 0.27 m. The total compacted to 0.20 m. After that, the 3.5-m-depth homogenous sand was placed at the bottom of the soil box and compacted to a height of 0.30 m. The upper layer of gravel around the specimen was placed and was moved into the soil box by a travelling crane and stood on the cap to monitor its displacement and rotation (Fig. 5(b and c)). Half of piles were instrumented with three pairs of DTs to obtain average pile curvatures at pile heads (Fig. 5(d)). The other half of piles were glued with 6 pairs of soil pressure cells (SPCs) on inner and outer surfaces at an interval of 1.0D = 15 cm (maximum embedded depth 6.0D = 90 cm) to record the soil pressure along different rows of piles, as shown in Fig. 5(e). In addition, two SPCs (embedded depths = 45 cm and 90 cm) and one SPC (depth = 60 cm) were glued on the East and North sides of soil box (Fig. 1), respectively, to verify the boundary condition. Fig. 5(f) illustrates the strain gauges (SGs) attached to the longitudinal rebars labeled “a” and “b” (see Fig. 5(a)). It is worth noting that the maximum embedded depth of SGs, i.e. 13.3D (200 cm), is sufficient to capture the underground pile damage, according to previous study by Wang et al. [20].

2.5. Loading scheme

In these tests, the lateral loading was displacement-controlled, with the protocol shown in Fig. 6. By connecting a 150 kN±250 mm-capacity hydraulic actuator with the cap, the lateral loading started at 5 mm and gradually increased until the end of loading (defined below). In each case, the lateral loading rate was as follows: before the displacement level of 10 mm, the loading rate was 0.5 mm/s, then increased to 1.0 mm/s until 80 mm, after which level the loading rate became 2.0 mm/s until end of loading. Each loading level was cycled three times.

3. Test results

3.1. Verification of boundary condition

To verify the boundary condition of soil box, Table 3 compares the soil pressure values on the outer face of pile #4 of S4 (nearest to the soil box wall, see Fig. 5(a)) and East wall of soil box (perpendicular to the loading direction, see Fig. 1(c)) at the same depth of 6.0D = 90 cm. It can be found that, the soil pressure on East wall of soil box was less than 10% of that on outer face of pile #4 before the displacement level of 130 mm. Additionally, in S6, the SPCs at depths ranging 45–90 cm glued on the outer surface of pile #6 did not work properly during tests. However, it is found that, with the loading displacement increases from 40 mm to 170 mm, the soil pressure at depth of 90 cm on the East wall of soil box only ranges from 4.07 kPa to 6.78 kPa, which is rather small. All the results indicate that the boundary condition is generally negligible though not fully eliminated.

3.2. Lateral load versus displacement

Fig. 7 presents the lateral load-deflection relationships. Generally, the plump hysteretic loops indicate ductile properties for the specimens. To be specific, for S1, peak values of lateral strengths were \(V_{max} = 19.37\) kN and \(V_{min} = -16.01\) kN in the push and pull direction, corresponding to displacement levels of 99.8 mm and –98.6 mm, respectively. For S4, maximum positive and negative lateral loads are \(V_{max} = 62.25\) kN and \(V_{min} = -55.05\) kN at 110.1 mm and –133.6 mm, respectively, which are much greater than S1 owing to increasing lateral stiffness. In S6, larger peak lateral forces \((V_{max} = 85.37\) kN and \(V_{min} = -86.92\) kN) are reached at displacement levels of 123.0 mm (push) and 113.0 mm (pull). Lateral strengths decrease gradually after reaching peak values in all cases. Specifically, the lateral forces at loading-ends show gradual degradation of 5.3% (18.3%), 22.5% (10.6%), and 16.7% (17.7%) in the push (pull) direction than corresponding peak strengths for S1, S4, and S6, respectively.

Note that in the tests, the lateral loadings ended when the lateral strength fell by 15% or irreparable pile damage emerged. However, the ultimate limit states of specimens discussed below are defined as either over 15% degradation of lateral strength or ultimate pile curvature \((\phi_u)\) is reached, whichever occurs first.

3.3. Observed pile damage

To investigate seismic failure mechanisms of the pile models, it is important to detect their damage during and after the tests.
3.3.1. Aboveground part

During and after the tests, physical damage to aboveground piles was detected, as presented in Fig. 8. The first concrete cracks were detected at side-pile heads at displacement levels of 30 mm, 20 mm, and 30 mm for S1, S4, and S6, respectively, as shown in Fig. 8(a)–(c). The concrete cover of the center-pile head in S6 also cracked at 40 mm. Then more visible cracks continued to occur and spread horizontally and diagonally. The inner-face concrete cover began to spall at 90 mm and 80 mm at side-pile heads in S4 and S6, respectively, as illustrated in Fig. 8(d and e). Definitions of face locations and No. of piles are shown in Fig. 5, the same below. Additionally, concrete cover spalling of center piles in S6 appeared at 120 mm (Fig. 8(f)), but less severe than side piles. At 160 mm, concrete core crushing was inspected at side-pile heads in S4 and S6, as shown in Fig. 8(h and i). However, concrete core crushing at side pile head of S1 was not found until end of loading (Fig. 8(g)). This may be caused by the difference of axial-load variations at side-pile heads during lateral loading, which will be simulated by numerical modeling in Section 4.3.

In summary, almost all aboveground damage was located in pile head regions, up to 1.0D length from the cap-pile interface, indicating the formation of aboveground plastic hinges.

3.3.2. Underground part

After tests, all specimens were excavated to detect pile damage below the soil surface. Fig. 9 depicts the final underground pile damage, in which embedded depths are marked for easy inspections. It is obvious that the failure mode was flexural for all specimens (i.e., with apparent horizontal cracks). Locations of underground plastic hinges in each specimen are summarized in Table 4. It can be found that (1) less severe
damage was observed in the center piles than that in the side piles; (2) the embedded depth of underground plastic hinge of side piles was increased by the increasing pile rows; (3) shallower plastic hinges were formed on the side piles than the center piles.

The first finding can be explained by the fact that during the test, side piles interchange between leading and trailing rows. The leading rows of piles suffer greater internal forces than the middle and trailing rows under lateral loading [20]. This explanation also applies to the third finding, which leads to a smaller bending moment gradient and thereby a greater depth-to-maximum-moment for the center piles, as compared to the side piles.

The second finding is induced by the effect of pile-group reduction. More rows of piles results in lower soil reaction applied on the leading piles, consequently leading to a smaller gradient of bending moment along piles, which thereby increases the depth-to-maximum-moment.

In Fig. 10, post-test elevation views of S4 and S6 are illustrated. The pinching of side piles was obvious for both specimens. The minimum net side-to-side spacing of adjacent rows of piles dropped to 21.6 cm and
24.5 cm from original 30 cm in S4 and S6, respectively, corresponding to embedded depths of 6.0D and 5.7D. The pinching of underground side piles is attributable to the facts that the leading piles suffer greater soil reaction than the trailing piles and that side piles interchange between leading and trailing rows under lateral cyclic loading [20], as mentioned before.

3.4. Curvature distribution along piles and yielding limit state determination

As mentioned above, leading piles bear larger lateral forces than center and trailing piles in the tests. Nevertheless, leading and trailing piles of S4 and S6 interchange during the cyclic loading. For better explanations, the leading piles mentioned below refer to those in the pull direction (i.e., pile #1 as shown in Fig. 5). Recall that the first-yielding state of the pile section is defined as when the first-yielding curvature (ϕy) is reached, which is ϕy = 0.0326 rad/m. Note that the minor impact of axial load variations on ϕy is detailed later in Section 4 “BNWF Model”.

Fig. 11 presents curvature distributions along piles in all cases, with which the yielding sequence of the specimens can be determined. Note that some SGs at pile heads malfunctioned during the tests and their values are estimated based on linear trends, as depicted by dashed lines in Fig. 11. Additionally, in S4, the SGs on piles below the embedded depth of 8D did not work properly and no strain data was recorded during the test, so pile curvature in this region (embedded depth = 8D – 13.3D) is not plotted in Fig. 11(b). Also, some abnormal curvature data (e.g., at depth of 90 cm or 105 cm for cases S1 and S6) are excluded from the drawing in Fig. 11. It can be found that pile heads yield before underground pile portions in all cases. More specifically, in S1 (Fig. 11(a)), the pile head yields before a displacement level of 50 mm. Combined with the testing phenomenon described above, 30 mm tends to be the first-yielding displacement of S1. Then, the first yielding of the underground part occurs at about 60 mm (embedded depth = 4D). In S4 (Fig. 11(b)), the pile head yields before a displacement level of 50 mm. Combined with the testing phenomenon described above, 30 mm tends to be the first-yielding displacement of S1. Then, the first yielding of the underground part occurs at about 60 mm (embedded depth = 4D). In S6 (Fig. 11(c and d)), the first-yielding of leading and center pile heads emerge at 40 mm and 50 mm, respectively. A larger depth-to-maximum-curvature of 5D is observed for this case. In S6 (Fig. 11(c and d)), the first-yielding of leading and center pile heads emerge at 40 mm and 50 mm, respectively. Then subgrade leading and center piles yield at 100 mm and 120 mm, respectively, with depths of maximum curvatures at 6D and 7D, respectively. It should be noted that the observed regions of severe pile damage match well with the recorded maximum pile curvatures, as shown in Fig. 11.

In addition, to quantitatively verify the similar restraint condition for cap/pile-head across different cases, Fig. 12 compares curvature distributions along aboveground piles of S1, S4, and S6 at a given small displacement level of 20 mm (before the yielding of pile head). It can be seen that the pile-head curvatures are non-zero, and more importantly they are quite close among the studied cases, indicating that the pile-
Soil Dynamics and Earthquake Engineering 132 (2020) 106074

Head restraint conditions are quite close across these cases (S1, S4, and S6) at the beginning of loading. In other words, the single pile case S1 can be approximately regarded as a single pile in pile-group cases of S4 and S6, but with neither pile-group effect nor cap-rotation effect. These effects will be thoroughly studied later in this paper.

4. BNWF model

4.1. Model description

Based on the BNWF method, FE models of soil-pile-interaction systems are developed in OpenSees (Fig. 13). The cap is modeled with elastic beam-column elements, and the piles are simulated using displacement-based beam-column elements with 5 integration points. The element lengths vary from 0.10 m to 0.20 m [51]. Note that horizontal and vertical translational degree-of-freedoms (DOFs) at pile tips (along y and z axes shown in Fig. 13(a)), perpendicular to the loading, are fixed for computational efficiency while those along to the loading direction and rotational DOFs are free to represent the boundary conditions in the tests. Additionally, to ensure the fixed pile-head/cap connection, the cap center in S1 is restrained against all rotational DOFs. Also noteworthy is that the P-∆ effect is considered in the FE models.

The pile section is discretized into 225 concrete fibers and 4 steel fibers. This fiber-discretization can produce stable curvature responses, as compared to a more refined discretization, for the adopted displacement-based beam-column element [52,53]. Concrete fibers adopt the Kent-Scott-Park [54] constitutive model, with confinement effect accounted [55]. Fig. 13(d) illustrates the cyclic behavior of steel. The longitudinal bar is modeled by the Parallel material consisting of the Steel 02 [56] and the Hysteretic material model for better convergence, as described in Wang et al. [20].

The soil reaction is modeled using discrete nonlinear zero-length elements, i.e., p-y springs [23], as shown in Fig. 13(a and e). The
American Petroleum Institute (API) method [57] is adopted to define p-y springs, as given in Eq. (2).

\[ p = Ap_u \tanh \left( \frac{n_h}{n_p} \right) \]  

where \( p \) = soil reaction at unit pile length (kN/m); \( A \) = loading factor (0.9 for cyclic loading); \( h \) = depth (m); \( p_u \) = ultimate bearing capacity at depth of \( h \) (kN/m); \( n_h \) = initial modulus of subgrade reaction (kN/m\(^3\)); \( y \) = lateral deflection (m). Note that the \( n_h \) value obtained by Chai and Hutchinson [17] for piles under large lateral deflections (similar to the situation in this study) is about 1/5–1/4 of the value recommended by ATC-32 [58] (Fig. 14(a)), not to mention the greater \( n_h \) in API [57]. Hence, the \( n_h \) is chosen to be 1/5 of the recommended value from ATC-32 [58]. The ultimate lateral resistance of soil, \( p_u \), can be calculated by Eqs. (3)–(5).

\[ p_u = (C_1h + C_2D)\gamma h \]  

\[ p_{ud} = C_3D\gamma h \]  

\[ p_u = \min\{p_u, p_{ud}\} \]  

where \( p_{ud} \) and \( p_u \) are the ultimate resistance of soil in the deep and shallow portion, respectively; \( C_1, C_2 \) and \( C_3 \) are coefficients determined by Fig. 14(b); \( \gamma \) = unit weight of soil (kN/m\(^3\)); \( D \) = width of pile (m); \( h \) = depth (m). It should be noted that although some concerns have been raised about the validity of the API method [30,59], it still gains recognition due to its convenient applications in the platform of OpenSees, which motivates the employment of this method in the present study.

In S4 and S6, PGE should be carefully considered. Since the leading and trailing rows of pile groups interchange during the reversed cyclic loading, the PGE is taken into account by multiplying the soil resistance of single pile p-y curves by an overall group reduction factor for all piles in the group [60–63]. In light of these studies, the group reduction factors in the present study are rigorously calibrated until well agreements between the numerical and experimental results are reached. To be specific, the group reduction factors are \( f_g = 0.61 \) and 0.40 for S4 and S6, respectively. For better readability, main parameters to define the numerical model are summarized in Table 5, all of which are calculated based on the physical properties of soils (Table 2) and pile dimensions (pile diameter \( D = 0.15 \) m, etc.). Specifically, \( A \) is suggested to be 0.9 for

![Fig. 13. Schematic illustrations of BNWF model: (a) 3D model; and constitutive models of (b) unconfined concrete; (c) confined concrete; (d) longitudinal steel; (e) p-y spring.](image)

![Fig. 14. Key parameters defining p-y springs:(a) Suggested \( n_h \) value from ATC-32 [58]; (b) Coefficients as function of \( \theta \) [57].](image)
cyclic loading [57]; C1, C2, and C3 are determined based on the friction angle \( \theta = 31^\circ \) according to Fig. 14(b); since \( D_i = 0.65\% \), the suggested \( n_0 \) value of ATC-32 [58] (Fig. 14(a)) is \( 11388.20 \text{kN/m}^3 \), and its 1/5 of this value, \( 2277.64 \text{kN/m}^3 \), is adopted according to Ref. [17], as discussed above; the embedded depth, \( h \), ranges from 0 to 3.5 m.

### 4.2. Model validation

Fig. 15 illustrates the lateral displacement-load relationships of test data and numerical simulation for all cases, together with the measured and calculated backbone curves of all specimens. In addition, comparisons of responses at different critical states between the BNWF models and tests are listed in Table 6. The errors fall into -10%~23%. Furthermore, Fig. 16 compares the experimental and numerical curvature distributions of all cases at the occurrence of equivalent yielding, with reasonably satisfactory prediction in the curvature values and locations of plastic hinges. In general, the good matches of hysteretic loops, backbone curves, responses at critical states, along with curvature distributions, between test and numerical results mean that the numerical model is validated and can be used to predict seismic behavior of the soil-pile system.

### 4.3. Variation of cross-section axial load for ultimate limit state estimate

As mentioned before, the cross-section ultimate curvature of piles (\( \phi_u \)) is probably dependent on the varied axial loads induced by the cap rotation. Therefore, the validated numerical model is required to determine the ultimate limit curvature by tracing the varied axial loads at potential locations of plastic hinges, such as pile heads and underground piles at the observed severe damage regions. Fig. 17(a) presents the development of cross-section axial compressive loads at leading pile heads during the tests. It can be seen that significant variations of axial loads are observed for the pile-group cases, as expected. By contrast, no axial load variations are detected in the single pile case (S1). More specifically, after reaching the equivalent-yielding limit states, axial loads in leading piles of S4 and S6 gradually increase to maximum values of 141.25 kN and 126.05 kN, respectively, and then drop to 125.83 kN and 109.99 kN, respectively. According to the above-described test phenomena, pile-head plastic hinges in the pile-group models should occur after 50 mm. Therefore, the cross-section axial loads when plastic responses occur should range from 125.83 kN to 141.25 kN and from 109.99 kN to 126.05 kN for S4 and S6, respectively. These two ranges correspond to relatively stable ultimate curvatures of 0.835 rad/m and 0.860 rad/m for S4 and S6, respectively.

Furthermore, as shown in Fig. 17(b), a parametric analysis is conducted in XTRACT [44] to reveal the sensitivity of axial load (P) on \( \phi_u \) and \( \phi_u \) values, which are critical to define the limit states of specimens. The axial load ratio, \( \alpha \), varies from -0.05 (tension) to 0.15 (compression). This large range is sufficient to cover the axial load variations in piles during the full cyclic tests. Obtained \( \phi_u \) and \( \phi_u \) values are summarized in Table 7. It is found that \( \phi_u \) is generally not sensitive to axial loads, even under such a large range of \( \alpha \) (coefficient of variance (COV) is as low as 3.85%). Therefore, a constant \( \phi_u \) of 0.0354 rad/m is adopted in this paper for convenience. However, \( \phi_u \) is strongly sensitive to axial loads (COV = 35.41%), especially under compression. This is the reason why FE models are developed to determine \( \phi_u \). Nevertheless, it should be noted that the studied range of axial loads in Fig. 17(b) is much larger than the abovementioned ranges of axial loads for S4 and S6 when plastic pile responses take place.

### 4.4. Ultimate limit state determination

Based on the above study, the ultimate limit states of all specimens can be determined. Fig. 18 illustrates the identification process of the ultimate limit states for all the specimens. Lateral displacements at the ultimate limit states are 120 cm, 160 cm, and 160 cm for S1, S4, and S6, respectively. Note that the lateral strength of specimen S1 degrades by 15% at the displacement of 120 cm, which is much before the curvature demand reaching its ultimate curvature. This is the reason why the displacement of 120 cm is taken as the ultimate limit state.

### 5. Discussions on seismic failure mechanisms

#### 5.1. Summary of plastic hinge formations and developments

To investigate seismic failure mechanisms of pile specimens with different pile layouts (1 × 1, 2 × 2, 2 × 3), the plastic hinge formations and developments are firstly revealed (see Fig. 19). Generally, each pile forms two plastic hinges in all cases: the first one at the pile head (limited to the 1.0D-length region below the cap-pile interfaces) and the second one beneath the ground. Specifically, in S1 (single pile), the

---

**Table 5**

Parameters for numerical modeling.

<table>
<thead>
<tr>
<th>Property (Unit)</th>
<th>Value</th>
<th>Property (Unit)</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cyclic loading factor, ( A )</td>
<td>0.9</td>
<td>Coefficient, ( C_2 )</td>
<td>2.089</td>
</tr>
<tr>
<td>Coefficient, ( C_2 )</td>
<td>2.804</td>
<td>Coefficient, ( C_3 )</td>
<td>32.515</td>
</tr>
<tr>
<td>Width of pile, ( D ) (m)</td>
<td>0.15</td>
<td>Group reduction factor, ( f_g )</td>
<td>0.61 (S4), 0.40 (S6)</td>
</tr>
<tr>
<td>Embedded depth, ( h ) (m)</td>
<td>0-3.5</td>
<td>Initial modulus of subgrade reaction, ( n_0 ) (kN/m(^2))</td>
<td>2277.64</td>
</tr>
<tr>
<td>Soil reaction at unit pile length, ( p ) (kN/m)</td>
<td>–</td>
<td>Ultimate bearing capacity, ( p_u ) (kN/m)</td>
<td>–</td>
</tr>
<tr>
<td>Ultimate resistance of soil in the deep portion, ( p_{ud} ) (kN/m)</td>
<td>–</td>
<td>Ultimate resistance of soil in the shallow portion, ( p_{us} ) (kN/m)</td>
<td>–</td>
</tr>
</tbody>
</table>
| Lateral deflection, \( y \) (m) | – | \[44\] to reveal the sensitivity of axial load (P) on \( \phi_u \) and \( \phi_u \) values, which are critical to define the limit states of specimens. The axial load ratio, \( \alpha \), varies from -0.05 (tension) to 0.15 (compression). This large range is sufficient to cover the axial load variations in piles during the full cyclic tests. Obtained \( \phi_u \) and \( \phi_u \) values are summarized in Table 7. It is found that \( \phi_u \) is generally not sensitive to axial loads, even under such a large range of \( \alpha \) (coefficient of variance (COV) is as low as 3.85%). Therefore, a constant \( \phi_u \) of 0.0354 rad/m is adopted in this paper for convenience. However, \( \phi_u \) is strongly sensitive to axial loads (COV = 35.41%), especially under compression. This is the reason why FE models are developed to determine \( \phi_u \). Nevertheless, it should be noted that the studied range of axial loads in Fig. 17(b) is much larger than the abovementioned ranges of axial loads for S4 and S6 when plastic pile responses take place.

4.4. Ultimate limit state determination

Based on the above study, the ultimate limit states of all specimens can be determined. Fig. 18 illustrates the identification process of the ultimate limit states for all the specimens. Lateral displacements at the ultimate limit states are 120 cm, 160 cm, and 160 cm for S1, S4, and S6, respectively. Note that the lateral strength of specimen S1 degrades by 15% at the displacement of 120 cm, which is much before the curvature demand reaching its ultimate curvature. This is the reason why the displacement of 120 cm is taken as the ultimate limit state.

5. Discussions on seismic failure mechanisms

5.1. Summary of plastic hinge formations and developments

To investigate seismic failure mechanisms of pile specimens with different pile layouts (1 × 1, 2 × 2, 2 × 3), the plastic hinge formations and developments are firstly revealed (see Fig. 19). Generally, each pile forms two plastic hinges in all cases: the first one at the pile head (limited to the 1.0D-length region below the cap-pile interfaces) and the second one beneath the ground. Specifically, in S1 (single pile), the
The equivalent-yielding limit states of aboveground and underground pile portions were reached at 30 mm and 60 mm, respectively. The underground plastic hinge was located at an embedded depth of \(4D\). S1 came into the ultimate limit state at 120 mm. In S4, the equivalent-yielding of pile heads and subgrade parts appeared at 50 mm and 90 mm (embedded depth of underground plastic hinge = \(5.3D\)), respectively. Then the ultimate limit state of S4 was achieved at 160 mm. S6 approximately equivalent-yielded at 50 mm and 90 cm on the top of side and center piles, respectively. Then, the equivalent-yielding of underground side and center piles emerged at 100 mm and 120 mm, respectively. Note that the underground plastic hinge on center piles, i.e., \(6.7D\), was slightly deeper than that on side piles (6D). S6 reached the ultimate limit state at 160 mm.

Compared with the single pile case S1 where cap rotation and PGE are not involved, the pile group cases show significantly different results on the displacement levels and locations of plastic hinges. Therefore, it is noted that the impact of cap rotation and PGE should be well considered when investigating seismic failure mechanisms of partially-embedded RC pile-group foundations.

### 5.2. Combined impact of cap rotation and pile-group effects

To investigate the combined effects of cap rotation and PGE on seismic failure mechanisms of pile-group specimens, lateral force-displacement curves from tests are studied herein. Fig. 20 shows the experimental backbone curves of S4 and S6. Note that, to highlight the combined impacts of cap rotation and PGE, two scenarios of four and six times of the experimental backbone curve of S1 are also presented, which should not exist in engineering practice. Since cap rotation is not involved in case S1, the “4 times S1 (Test)” scenario in Fig. 20(a) represents the assumed condition that both the cap rotation and PGE are ignored when loading on S4. The load gap between “S4 (Test)” and “4
times S1 (Test)” highlights the combined impacts of cap rotation and PGE in loading tests. Similar relationship is applied to scenarios of “S6 (Test)” and “6 times S1 (Test)”. More specifically, it can be found that the experimental lateral loads of S4 and S6 are always lower than the four- and six-time results of S1, with an average bias of 18.2% and 12.7%, respectively, before reaching the peak-strength limit state of S1.
(i.e., at the displacement level of 100 mm). The comparison of each limit state in different scenarios are summarized in Table 8. It is found that, at each limit state, the experimental lateral loads of S4 (S6) are -24.9% (~5.9% ~-22.3%~0.1%) lower than the four-time (six-time) results of S1. In addition, the lateral displacements at individual limit states in S4 and S6 are 10.0% and ~66.7% greater than that in S1, which are caused by the combined effects of cap rotation and PGE. Note that, at equivalent yielding limit state, the differences of lateral displacements between scenarios of “S4” (S6) and “four-time of S1” (six-time of S1) are much significant than the corresponding lateral load differences, reflecting the complex roles of cap rotation and PGE. The pile-group cap rotation induces varied axial loads between different rows of piles, which will not occur in single-pile case S1 (Fig. 17(a)). It is deduced that lateral load increases lead to significantly increased flexural stiffness, but the growth of leading-pile bending moments are minor, consequently, leading-pile-head curvature in S4/S6 are much lower than that in S1. Since $\phi_e$ is stable when axial load varies (Fig. 17(b)), the equivalent-yielding lateral displacements of S4/S6 are larger than S1. It is also deduced that, among the two impacts, PGE is dominant over cap rotation on the lateral strength of pile-soil systems. Before the pile-head yielding, the pile-soil system remains elastic, leading to limited impact of PGE, hence the corresponding lateral loads between S4 (S6) and their respective counterparts are much lower. Nevertheless, all these deductions need further demonstration by numerical simulation.

5.3. Separate impact of cap rotation

To further reveal the separate effect of cap rotation, the validated BNWF model is adopted, in which the cap-orthocenter is modeled as rotational free or fixed (along y axis shown in Fig. 13(a)) for scenarios with or without cap rotation, respectively. Fig. 21 illustrates the comparison of numerical backbone curves between S1 and S4 (S6). Note that PGE is not considered by setting a group reduction factor of 1.0 for the pile-group cases. It can be found in both subfigures that, in the scenarios without cap rotation, the lateral loads of S4 and S6 at any given deflection are equal to four and six times of S1, respectively, with the same corresponding displacements at individual limit states. These findings verify the adopted test technique that the fixed-head single pile can be seen as a pile extracted from pile groups, in which both the cap rotation and PGE are not considered. Meanwhile, it can also be found that, the presence of cap rotation of S4 and S6 significantly increases the lateral displacements at individual limit states. For better readability, Table 9 compares the seismic response at each limit state in different scenarios of S4/S6 with or without cap rotation. More specifically, the displacement differences at individual limit states in S4 (between the case with and without cap rotation) are greater than those in S6. For instance, with cap rotation considered, the displacements of S4 corresponding to “Equivalent-yielding”, “Peak strength”, and “Ultimate” limit states increase by 56.1%, 10.0%, and 28.0% than their counterparts, similarly, the corresponding displacements of S6 at those three states rise by 7.6%, 11.1%, and 7.7%, respectively. However, neither the occurrence sequence nor the corresponding lateral strength at each limit state (bias = -0.4%~2.4%) is obviously changed.

To investigate the separate impact of cap rotation on seismic performance of pile groups, it is necessary to obtain the distributions of axial and lateral loads between each row piles. Table 10 summarizes the distributions of axial and lateral loads (at pile heads) between every row piles of S4/S6 in scenarios with or without cap rotation at 30 mm. It indicates that, with cap rotation considered, leading-row piles of S4/S6 suffer greater axial and lateral loads than middle-row and trailing-row piles. However, the axial and lateral loads applied on each row piles of S4/S6 are equal in scenarios without cap rotation. Considering this, side piles of pile groups in the scenario with cap rotation are more prone to undergo plastic damage when subjected to cyclic lateral loading.

Fig. 20. Experimental backbone curves of specimens (a) S4 and (b) S6.

### Table 8

<table>
<thead>
<tr>
<th>Case</th>
<th>Equivalent yielding limit state</th>
<th>Peak-strength limit state</th>
<th>Ultimate limit state</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\Delta_{y,e}$ (mm)</td>
<td>$F_{y,e}$ (kN)</td>
<td>$\Delta_{y,p}$ (mm)</td>
</tr>
<tr>
<td>S1 (Test)</td>
<td>30.0</td>
<td>12.02</td>
<td>100.0</td>
</tr>
<tr>
<td>4 times S1 (Test) (1)</td>
<td>30.0</td>
<td>48.08</td>
<td>100.0</td>
</tr>
<tr>
<td>S4 (Test) (2)</td>
<td>50.0</td>
<td>45.22</td>
<td>120.0</td>
</tr>
<tr>
<td>Difference = [(2)-(1)]/ (1)</td>
<td>66.7%</td>
<td>-5.9%</td>
<td>20.0%</td>
</tr>
<tr>
<td>6 times S1 (Test) (3)</td>
<td>30.0</td>
<td>72.12</td>
<td>100.0</td>
</tr>
<tr>
<td>S6 (Test) (4)</td>
<td>50.0</td>
<td>72.07</td>
<td>110.0</td>
</tr>
<tr>
<td>Difference = [(4)-(3)]/ (3)</td>
<td>66.7%</td>
<td>-0.1%</td>
<td>10.0%</td>
</tr>
</tbody>
</table>

$\Delta_{y,e}, F_{y,e}$ - lateral displacement and load corresponding to equivalent-yielding limit state, respectively.

$\Delta_{y,p}, F_{y,p}$ - lateral displacement and load corresponding to peak-strength limit state, respectively.

$\Delta_{y,u}, F_{y,u}$ - lateral displacement and load corresponding to ultimate limit state, respectively.
that, in the scenario without cap rotation, the curvature distribution is always the same for S4 and S6. Meanwhile, with cap rotation considered, lower values of maximum curvature in underground piles are found (Fig. 22), meaning that the underground pile yielding and plastic hinge formation is delayed. In addition, compared with S4, the pile curvature distribution of S6 tends to be less affected by the effect of cap rotation due to its greater rotational stiffness. That is the reason why greater displacement differences at individual limit states are observed in S4 than S6 (Fig. 21). In summary, the separate effect of axial load variations primarily changes the displacement levels at individual limit states, while slightly affects the lateral strength at individual limit states.

5.4. Separate impact of pile group effects

To study the separate impact of PGE on seismic failure mechanisms, the validated numerical model is again employed. Note that the cap-orthocenter is fixed against rotation to neglect the impact of cap rotation herein. As mentioned before, the group reduction factors are set to be 0.61 and 0.40 for all piles in S4 and S6, respectively, to consider PGE. No group reduction factor is utilized for the reference scenarios without PGE. Fig. 23 presents the backbone curves of S4 and S6 with and without PGE. Lateral displacements and loads corresponding to different limit states of S4 and S6 with and without PGE are listed in Table 11.

It can be seen that, without PGE, the lateral strengths at individual limit states of S4 and S6 are enhanced obviously, meanwhile, significant decreases in the corresponding displacements are obtained. As shown in Table 11, in comparison with the scenarios without PGE, the average decreases of lateral strength are 11.3% and 19.7% for S4 and S6, respectively, while the corresponding displacements rise by 15.8% (S4) and 33.4% (S6) averagely when PGE is considered. Another finding is that higher differences of lateral loads and displacements are obtained in S6 than S4. For instance, compared with the scenarios without PGE, peak lateral strengths are dropped by 11.5% and 21.6% for S4 and S6, respectively, meanwhile, the lateral displacements are increased by 11.1% and 22.2% for S4 and S6, respectively.

This finding lies in that the soil reaction on piles is reduced when PGE is considered, leading to lower lateral stiffness of the soil-pile systems and smaller gradients of bending moment along piles.

6. Conclusions

Pile foundations, especially partially-embedded ones, are the seismic...
vulnerable parts of bridges, which has drawn much attention. In this study, a series of quasi-static tests were conducted on three pile specimens with different pile layouts ($1 \times 1$, $2 \times 2$, $2 \times 3$). Then the BNWF method was adopted and validated to reveal seismic failure mechanisms of partially-embedded RC pile-group foundations in sand and to study the combined/separate impact(s) of cap rotation and PGE. Based the work in this paper, main conclusions can be drawn:

1. Flexural failure modes are detected for partially-embedded pile-group foundations under lateral loadings. Side pile heads yield firstly in all pile specimens, forming the first plastic hinge below the pile-cap interface up to $1.0D$ length, followed by the yielding of center pile heads. Subgrade plastic hinges tend to occur after the aboveground ones.

2. For pile groups under cyclic loading, the cap rotation can induce axial load variations on piles. The ultimate curvature of the pile section is strongly dependent on the varied axial loads while the equivalent-yielding curvature is not. The role of cap-rotation effect can significantly cause lateral displacements at individual limit states of pile groups increase by maximum value of 56.1%. This trend is more obvious in $2 \times 2$ pile group than $2 \times 3$ pile group. However, the role of cap rotation hardly affects the corresponding lateral loads at these limit states. The above finding implies a future direction to control cap rotation under lateral loads, which can control the ductile behavior of pile group foundations in design.

3. The role of pile group effect can increase lateral displacements at individual limit states by as large as 39.4%, while reduce lateral
Table 11

<table>
<thead>
<tr>
<th>Case</th>
<th>Equivalent-yielding limit state</th>
<th>Peak-strength limit state</th>
<th>Ultimate limit state</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Δy,Δu, Fy,e (mm)</td>
<td>Δy,Δu, Fy,p (kN)</td>
<td>Δy,Δu, Fy,u (kN)</td>
</tr>
<tr>
<td>S4 without PGE</td>
<td>33</td>
<td>59.04</td>
<td>90</td>
</tr>
<tr>
<td>S4 with PGE</td>
<td>39.5</td>
<td>54.11</td>
<td>100</td>
</tr>
<tr>
<td>S6 without PGE</td>
<td>19.7% −8.4%</td>
<td>11.1% −11.5%</td>
<td>16.7% −14.1%</td>
</tr>
<tr>
<td>S6 with PGE</td>
<td>46</td>
<td>74.3</td>
<td>112.97</td>
</tr>
<tr>
<td>Difference</td>
<td>39.4% −16.1%</td>
<td>22.2% −21.6%</td>
<td>38.5% −21.5%</td>
</tr>
</tbody>
</table>

Δy,Δu, Fy,e - lateral displacement and load corresponding to equivalent-yielding limit state, respectively.

Δy,Δu, Fy,p - lateral displacement and load corresponding to peak-strength limit state, respectively.

Δy,Δu, Fy,u - lateral displacement and load corresponding to ultimate limit state, respectively.

loads at these limit states by up to 21.6%. This influence is more significant in 2 × 3 pile group than 2 × 2 pile group. Gathering the above second conclusion, it is reasonable to infer that with the increasing number of row in pile-groups, contribution of the pile group effect tends to increase while that of the pile-cap rotation reduces.

Declaration of competing interest

Declarations of interest: none.

CRediT authorship contribution statement

Tengfei Liu: Conceptualization, Methodology, Investigation, Formal analysis, Software, Visualization, Validation, Writing - original draft, Writing - review & editing. Xiaowei Wang: Conceptualization, Funding acquisition, Methodology, Software, Visualization, Writing - original draft, Writing - review & editing. Aijun Ye: Conceptualization, Supervision, Funding acquisition, Resources, Writing - original draft, Writing - review & editing.

Acknowledgment

The authors gratefully acknowledge the financial support provided by the National Natural Science Foundation of China (Grant No. 51778469) and the State Key Laboratory of Disaster Reduction in Civil Engineering, Ministry of Science and Technology of China (Grant No. SDLRCE19-B-20). The first author truly appreciates the financial support from the China Scholarship Council. The second author appreciates the assistance of Prof. Jianzhong Li and Mr. Chuan’an Lu. The constructive comments from Prof. Salgado (Purdue University), Dr. Yu Shang, Mr. Lianxu Zhou, and Mr. Deming Zhang to improve the manuscript are also appreciated.

Appendix

List of Symbols

- D: Width of pile section
- α: Axial compressive ratio of pile section
- P: Axial load of pile section
- Ag: Gross area of pile section
- fc: Uniaxial strength of concrete
- fy, fy: Yield stress of longitudinal steel and transverse bar
- fu: Ultimate stress of longitudinal steel
- ea, eu: Ultimate strain corresponding to fa, eu
- σy, σu: Ultimate strain of concrete core
- φy, φu: Ultimate curvature of pile section
- Vp: Transverse volumetric ratio of pile
- m: Average grain size
- Dr: Relative density of sand
- θ: Friction angle of sand
- γ: Dry density of sand
- Ci, Cc: Coefficients of uniformity and curvature
- εmax, εmin: Maximum and minimum void ratios of sand
- e: Void ratio of sand
- Vmax, Vmin: Maximum and minimum lateral loads on specimen
- Δ: Lateral loading displacement
- p: Soil reaction at unit pile length
- y: Lateral deflection of pile
- A: Loading factor
- h: Depth
- f: Group reduction factor
- pu: Ultimate bearing capacity at depth of h
- m: Initial modulus of subgrade reaction
- psoil, pad: Ultimate resistance of soil in the shallow and deep portions
- Ci, Cc: Coefficients of determining ps and pad
- Δy, Δu, Fy,e: Lateral displacement and load corresponding to peak-strength limit state
- Δy, Δu, Fy,p: Lateral displacement and load corresponding to peak-strength limit state
- Δy, Δu, Fy,u: Lateral displacement and load corresponding to ultimate limit state

References

T. Liu et al.

17


