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Some Observations of the Influence Factors on the Response of Pile Groups

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Abstract

A simplified approach for nonlinear analysis of the load-displacement response of pile groups embedded in multilayered soils is presented in this work. A hyperbolic model is used to capture the relationship between unit skin friction and pile-soil relative displacement developed along the pile-soil interface and the stress-displacement relationship developed at the pile end. Considering interactive effect among piles, the parameters related to the hyperbolic model of an individual pile in a group can be computed. As to the analysis of the response of pile groups, a highly effective iterative computer program is developed using the hyperbolic model of an individual pile in a group. The efficiency and accuracy of the present method is verified using a well-documented field test. Furthermore, a parametric study is conducted to capture the influence of pile spacing and number of piles on the load-settlement response of the pile groups connected to a rigid cap. The pile-group settlement ratio and the pile-group resistance ratio are analyzed to assess the interaction effect among individual piles.

Keywords: *single pile, pile groups, skin friction, end resistance, settlement, a hyperbolic model*

1. Introduction

In the past several years, a number of theoretical methods have been made to analyze the behavior of single pile and pile groups. Many of the various numerical methods fall into three main categories: (1) the theoretical load-transfer curve method (Coyle and Reese, 1966; Kraft *et al.*, 1981; Armaleh and Desai, 1987; Hirayama, 1990; Zhang *et al.*, 2010), which uses different load-transfer functions to describe the relationship between unit skin friction and pile-soil relative deformation. However, it can not be directly used to capture the response of pile groups because the interactive effect of soil movement surrounding the pile shaft can not be considered in the load-transfer curve method. (2) the shear displacement method (Randolph and Wroth, 1978; Lee, 1993; Guo and Randolph, 1999; Yang *et al.*, 2011), which uses a logarithmic relationship to simulate the soil deformation induced by the shaft shear stress. However, the interaction between two continuous soil layers can not be taken into account in the shear displacement method. (3) the numerical method, including finite element method (Maharaj and Gandhi, 2004; Comodromos *et al.*, 2009) and boundary element method (Mendonca and De Paiva 2000; Ai and Han, 2009), which is considered as one of the most powerful techniques for the analysis of the response of single pile and pile groups. However, the application of numerical methods is limited in practical application because of complex modeling procedures and high computational requirements, especially for large pile groups.

The distribution of load in pile groups is different from that in single pile due to complex interactions of piles, pile caps, and surrounding soils. Therefore, the interactive effect among piles should be taken into account when the calculated approaches for the single pile response are extended to the analysis of the behavior of pile groups. To account for the interactive effect among piles, the interaction factor defined for two equally loaded identical piles as the ratio of the increase in settlement of a pile due to an adjacent pile to the settlement of a single pile due to its own load was first introduced by Poulos (1968), who proposed that pile group effects can be assessed by superimposing the effects of two piles. Subsequent numerous studies (Poulos and Mattes, 1971; Lee, 1993; Zhang *et al.*, 2010) have been carried out using the simplified concept of interaction factor. It has been recognized that the conventional interaction factor approach tends to exaggerate the interactive effect of pile groups,

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thereby leading to an overestimation of the pile movement, as reported by Southcott and Small (1996), Mylonakis and Gazetas (1998), and Chen *et al*. (2011). Therefore, for the further prediction of pile group settlement, it is necessary to revisit the interaction factor problem between two vertically loaded piles.

In practical applications, the load-transfer approach presented by Coyle and Reese (1966) is an efficient method for the analysis of the single pile response. In their method, the load-transfer function is required to describe the relationship between mobilized unit skin friction and pile movement. Such a load-transfer function concept was first developed by Seed and Reese (1957), and later extended by other researchers (Armaleh and Desai, 1987; Hirayama, 1990; Lee and Xiao, 2001; Zhang and Zhang, 2012a). To account for the non-linearity in the stress-displacement response of soil, a hyperbolic model is commonly used to capture the relationship between unit skin friction and pile-soil relative displacement developed along the pile-soil interface and the stress-displacement response developed at the pile base. However, the conventional load-transfer approaches are rather difficult to extend to the analysis of the response of pile groups.

Field test is considered as the most reliable means of assessing the response of pile groups. However, field test is commonly expensive and time-consuming. Consequently, a simple method used to rapidly estimate the response of pile groups and to analyze nonlinear response of pile shafts and surrounding soils will be beneficial. The objective of the present paper is to obtain a rapid predication on the response of pile groups using load transfer method and shear displacement method. In this proposal, a hyperbolic model is used to capture the relationship between unit skin friction and pile-soil relative displacement developed along the pile-soil interface and the stress-displacement response developed at the pile end. Considering the interactive effect among individual piles, the values of the parameters related to the hyperbolic model of skin friction and end resistance of an individual pile in a group are computed. As to the analysis of the response of pile groups, a computational flow chart is developed. A parametric study is then conducted to capture the influence of pile spacing and number of piles on the load-settlement response of the pile groups connected to a rigid cap. To assess the interaction effect among individual piles, the pile-group settlement ratio and the pile-group resistance ratio are analyzed.

2. Hyperbolic Model of Skin Friction

The relationship between unit skin friction and its corresponding shear deformation can be approximated by a hyperbolic equation. One obtains:

$$\tau_s(z) = \frac{S_s(z)}{a + bS_s(z)} \tag{1}$$

where, $\tau_s(z)$ is the shear stress at a given depth z; a and b are empirical coefficients; and $S_s(z)$ is the relative deformation along the pile-soil interface at a given depth z.

As suggested by Randolph and Wroth (1978), the value of a can be taken as the reciprocal of the spring stiffness of soils around pile shaft, k_s. One obtains:

$$a = \frac{1}{k_s} = \frac{r_0}{G_s}\ln\left(\frac{r_m}{r_0}\right) \tag{2}$$

where, r_0 is the pile radius; G_s is the soil shear modulus; and r_m is the radial distance from the pile center to a point where the shaft shear stress caused by the pile can be neglected. In homogenous soils the value of r_m can be adopted as (Randolph and Wroth, 1978):

$$r_m = 2.5L(1 - \nu_s) \tag{3}$$

In multilayered soils, a modified expression for r_m can be written as:

$$r_m = 2.5L\frac{\sum_{i=1}^{n_s} G_{si}L_i}{G_{sm}L}\left(1 - \frac{\sum_{i=1}^{n_s} \nu_{si}L_i}{L}\right) \tag{4}$$

where, ν_{si} is the Poisson's ratio of soil layer i around pile shaft; G_{si} is the shear modulus of soil layer i around pile shaft; L_i is the thickness of soil layer i; G_{sm} is the maximum shear modulus in the soil layers; L is the pile length; and n_s is the number of soils.

It's well known that the loaded piles are highly nonlinear, whereas those for the adjacent load-free piles are linearly elastic. Therefore, the interaction between piles is commonly assumed to be in a linear elastic state, and Eq. (3) or Eq. (4) can be assumed to be valid for the soil with nonlinearly elastic behavior and the slippage interface between pile and soil.

The reciprocal of coefficient b can be taken as the unit skin friction at a very large value of the pile-soil relative deformation. Following suggestion of Zhang *et al*. (2014), the value of b can then be calculated by:

$$b = \frac{R_{sf}}{K_0\left(\frac{K}{K_0}\right)\tan\left[\varphi\left(\frac{\delta}{\varphi}\right)\right]\sigma'_{vz}} \tag{5}$$

where, K is the lateral earth pressure coefficient; K_0 is the coefficient of horizontal earth pressure on the pile for the soil layer; R_{sf} is in the range 0.8 to 0.95; σ'_{vz} is the effective overburden pressure at the depth under consideration; φ is the angle of shearing resistance of the soil; and δ is the friction angle of the pile-soil interface. Suggested values of lateral earth pressure coefficient, K, and friction angle of the pile-soil interface, δ, are summarized by the first author (Zhang and Zhang, 2012b) in another paper.

2.1 Hyperbolic Model of End Resistance

A hyperbolic model can be used to simulate the relationship between unit end resistance and pile base displacement. This hyperbolic relationship can be described by the following equation:

$$q_b = \frac{S_b}{f + gS_b} \tag{6}$$

where, q_b is the unit end resistance; f and g are empirical coefficients commonly determined experimentally or by back-analysis of field test results; and S_b is the pile base load.

The value of f can be taken as the reciprocal of the initial gradient of the base response, k_b, and may be conveniently expressed using the following equation (after Randolph and Wroth 1978):

$$f = \frac{1}{k_{\mathrm{b}}} = \frac{\pi r_0 (1-\nu_{\mathrm{b}})}{4G_{\mathrm{b}}} \tag{7}$$

where, G_b and ν_b are the shear modulus and Poisson's ratio of the soil below the pile base, respectively. The reciprocal of coefficient g can be taken as the unit end resistance at a very large value of the pile base deformation. One obtains:

$$g = \frac{R_{\mathrm{bf}}}{q_{\mathrm{bu}}} \tag{8}$$

where, R_{bf} is a failure ratio of end resistance. The ultimate toe stress, q_{bu}, can be calculated in the following form:

$$q_{\mathrm{bu}} = cN_{\mathrm{c}} + \bar{q}N_{\mathrm{q}} \tag{9}$$

where, c is the cohesion of soil supporting the pile; $\bar{q} = (1+2K_0)\sum\gamma_i L_i/3$; and γ_i is the unit weight of soil layer i. N_{c} and N_{q} are the bearing capacity factors and can be calculated with Eqs. (10) and (11), respectively. One obtains:

$$N_{\mathrm{c}} = (N_{\mathrm{q}} - 1)\cot\varphi \tag{10}$$

$$N_{\mathrm{q}} = (\tan\varphi + \sqrt{1+\tan^2\varphi})^2 e^{2\psi\tan\varphi} \tag{11}$$

where, ψ is the angle in radians for the failure surface at the pile base, which varies from 60° for soft clay to approximately 105° for dense sandy soils.

3. Analysis of Response of Pile Groups

3.1 Parameters Related to the Hyperbolic Model of Skin Friction of an Individual Pile in a Group

Considering interactive effect among piles, the vertical deformation of individual pile i is composed of three parts: (1) the settlement induced by its own loading; (2) the displacement induced by the neighbouring loaded piles; and (3) the deformation caused by the reinforcing effect of adjacent load-free piles.

The shaft shear stress at a given depth is assumed to be the same for all piles in a group. Considering interactive effect among piles, the total equivalent spring stiffness of the soils around pile i, k_{si}, can be expressed as:

$$\frac{1}{k_{\mathrm{s}i}} = \frac{1}{k_{\mathrm{s}ii}} + \frac{1}{k_{\mathrm{s}ij}} - \frac{1}{k'_{\mathrm{s}ij}} \tag{12}$$

where, k_{sii} is the spring stiffness of soils around pile i due to its own loading, which can be calculated using Eq. (2); k_{sij} is the variation of spring stiffness of the soils around pile i caused by the shear stress of the neighbouring loaded piles; and k'_{sijj} is the variation of spring stiffness of the soils around pile i due to the reinforcing effect of the adjacent load-free piles.

The values of k_{sij} and k'_{sijj} can be computed by (Zhang *et al.*, 2014):

$$k_{\mathrm{s}ij} = \sum_{j=1, j\neq i}^{n_{\mathrm{p}}} \frac{G_{\mathrm{s}}}{r_0 \ln\left(\frac{r_{\mathrm{m}}}{r_{ij}}\right)} \tag{13}$$

$$k'_{\mathrm{s}ij} = \sum_{j=1, j\neq i}^{n_{\mathrm{p}}} \frac{G_{\mathrm{s}} r_{ij}}{r_0^2 \ln\left(\frac{r_{\mathrm{m}}}{r_{ij}}\right)} \tag{14}$$

where, r_{ij} is the center to center distance between pile i and pile j.

The value of a_{g} can be taken as the reciprocal of the initial elastic soil stiffness along the pile-soil interface of individual pile i. The value of the parameter, b_{g}, related to the hyperbolic model of skin friction of an individual pile i in pile groups can be taken as identical to the value of b of a single pile.

3.2 Parameters Related to the Hyperbolic Model of End Resistance of an Individual Pile in a Group

For a group of n_{p} piles, the interactive effect of the displacement induced on the base of pile i can be established by using the principle of superposition. The displacement at the base of pile i consists of two parts: (*i*) the pile-end settlement due to its own loading; and (*ii*) the base displacement induced by the vertical load developed at the base of the adjacent loaded piles.

Assume the end resistance is the same for all piles in the n_{p}-pile group. The total equivalent soil stiffness at the base of pile i, k_{bi}, can be calculated by:

$$\frac{1}{k_{\mathrm{b}i}} = \frac{1}{k_{\mathrm{b}ii}} + \frac{1}{k_{\mathrm{b}ij}} \tag{15}$$

where, k_{bii} is the soil stiffness at the base of pile i induced by its own loading and can be derived from Eq. (7); and k_{bij} is the variation of soil stiffness at the base of pile i caused by the end resistance of the adjacent loaded piles, which can be computed by:

$$k_{\mathrm{b}ij} = \frac{2\pi G_{\mathrm{b}}}{(1-\nu_{\mathrm{b}}) \sum_{j=1, j\neq i}^{n_{\mathrm{p}}} \frac{1}{r_{ij}}} \tag{16}$$

The value of f_{g} can be taken as the reciprocal of the initial elastic soil stiffness at the base of individual pile i. The value of the parameter, g_{g}, related to the hyperbolic model of end resistance of *i*ndividual pile i in pile group can be taken as identical to the value of g of a single pile.

3.3 Computational Flow Chart for the Analysis of the Response of Pile Groups

Based on the computational flow chart shown in Fig. 1, the load-settlement response of an arbitrary pile in pile groups can be obtained. After an arbitrary pile response is clarified, the response of a pile group can also be assessed. Therefore, the present iterative computer program is suitable for analyzing the response of a pile group and a single pile response. Assuming a series of different values of pile end settlement, S_{bn}, a load-displacement curve can be obtained using the computational flow chart.

3.4 Comparison of Computed Results with Field Measured Results for Pile Groups

One case history reported in literature performed on pile groups is used to check the reliability of the present method. A loading test on the 3 × 3 pile groups embedded into multilayered soils was presented by Koizumi and Ito (1967). Nine piles were placed at a spacing of 3 times pile diameter and connected to a rigid cap in contact with the soil. The piles were 0.3 m diameter tubular steel pipes with a wall thickness of 3.2 mm. The pile length was 5.5 m, and the elastic modulus of the steel piles was adopted as 200 GPa. The subsoil was composed of two soil layers, with the sandy silt extending to a depth of 1.7 m and the silty clay below the depth of 1.7 m. Based on the back-analysis of the single pile behavior deduced from the loading tests, Cairo and Conte (2006) presented that the values of the elastic modulus of sandy silt and silty clay were about 12.8 MPa and 15.6 MPa, respectively. The back-analyzed value of the ultimate bearing capacity of the soil at the pile base was adopted as 25 kN as suggested by Cairo and Conte (2006). In this calculation, an average value of τ_{sf} of the sandy silt is adopted as 25 kPa, whereas an average value of τ_{sf} of the silty clay extending to a depth of 5.5 m is taken as 30 kPa. Poisson's ratio of the two-layer subsoil is assumed to be 0.5. The value of R_{sf} is adopted as 0.90 for the whole deposit, whereas the value of R_{bf} is assumed to be 0.90.

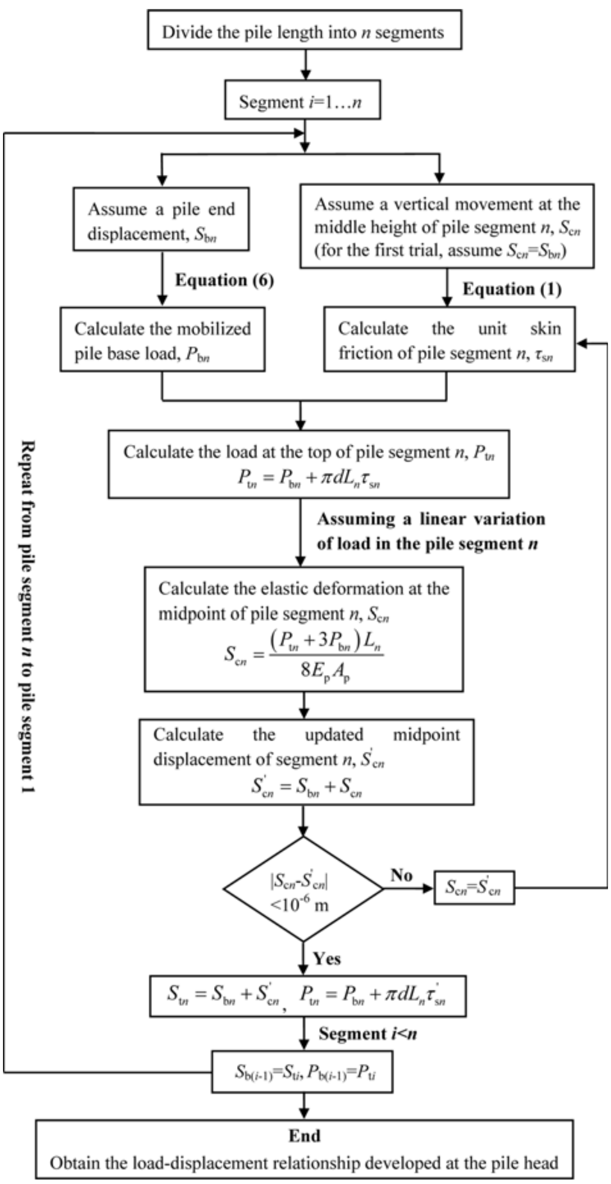

Fig. 1. Computational Flow Chart for the Analysis of the Response of a Single Pile or an Individual Pile in a Group

The measured results of the pile groups given by Koizumi and Ito (1967) are compared in Fig. 2 with that theoretically predicted using the present solution and the computed values derived from Cairo and Conte (2006).

Figure 2 shows that there is a great discrepancy between the computed results proposed by Cairo and Conte (2006) and the calculated values derived from the present method, especially at very high load levels. There is a large difference between the computed results proposed by Cairo and Conte (2006) and the measured results given by Koizumi and Ito (1967) during the entire load. However, the load-settlement relationship at the pile head plotted from the present method is generally consistent with the measured results given by Koizumi and Ito (1967) during the entire load. The reliability of the proposed method for the analysis of the load-settlement response of pile groups can be verified. It also can be investigated that the settlement of the pile groups derived from the present method increases with an increase in the value of R_{sf} at the same loading level.

3.5 Parameter Study on the Response of Pile Groups Connected to a Rigid Cap

In practice, it will be very complicated to account for the

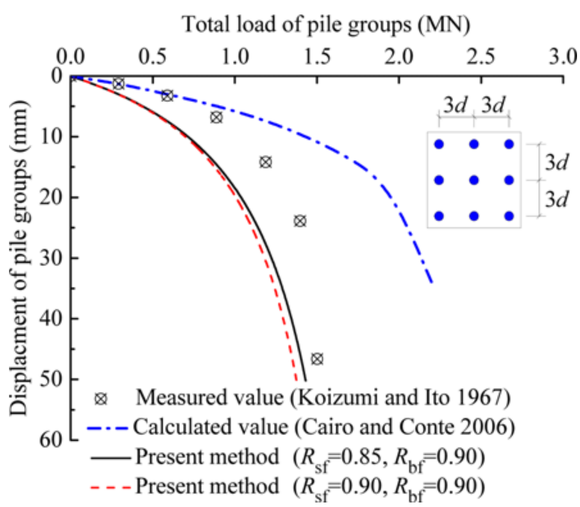

Fig. 2. Measured and Calculated Load-Settlement Curves of the Pile Groups Connected to a Rigid Cap

influence of the raft or pile-cap rigidity on the response of pile groups. To be simplified, herein the pile groups connected to a rigid cap is taken into account to capture the influence of pile spacing and number of piles on the behavior of pile groups. Furthermore, to assess the interaction effect among individual piles, the pile-group settlement ratio and the pile-group resistance ratio are analyzed.

The following cases are carried out to evaluate the influence factors of the response of pile groups. The individual pile length is adopted as 30 m, and the pile diameter is taken as 1 m. The pile with Young's modulus of 33 GPa is installed in a three-layer soil profile. The thickness of each soil layer is 10 m with the τ_{sf} value of 30 kPa, 40 kPa and 50 kPa for the upper, middle and lower soil layer, respectively, whereas the shear modulus of each soil layer, G_s, is taken as 10 GPa, 15 GPa and 20 GPa from the pile head to end, respectively. The shear modulus of the soil below the pile base, G_b, is assumed to be 30 GPa, and the ultimate unit end resistance, q_{bu}, is adopted as 1000 kPa. Based on the above-mentioned parameters, the behaviors of the 2×2 pile groups and 3×3 pile groups with pile spacing of $3d$, $4d$, $6d$ and $10d$ are analyzed using the proposed computational flow chart, respectively.

Before the analysis of the response of pile groups is conducted, the influence of the failure ratio of skin friction and end resistance, R_{sf} and R_{bf}, on the single pile behavior should be clarified. In the analysis of a single pile response, the Poisson's ratio of the three-layer soil is assumed to be 0.35. The values of R_{sf} and R_{bf} are adopted as 0.85, 0.90 and 0.95 for the whole deposit around pile shaft and the soil below the pile toe, respectively. The load-settlement response of the single pile can be calculated using the present iterative computer program. The influence of R_{sf} and R_{bf} on the load-displacement curve of the single pile is shown in Fig. 3.

Figure 3 shows that at low loading level, the parameters R_{sf} and R_{bf} almost have no effects on the single pile response. The influence of R_{sf} and R_{bf} on the load-displacement curve of the single pile gradually develops at high loading level, and the pile head settlement increases with increasing values of R_{sf} and R_{bf} at the same loading level.

3.6 Influence of Pile Spacing on Response of Pile Groups Connected to a Rigid Cap

Using the above-mentioned cases, the pile spacing is selected as $3d$, $4d$, $6d$ and $10d$ to capture the influence of pile spacing on the response of the nine-pile group connected to a rigid cap. In the calculation, Poisson's ratio of the whole deposit is assumed to be 0.35. The values of R_{sf} and R_{bf} are adopted as 0.90 for the soil around pile shaft and below the pile base, respectively. The behaviors of the 9-pile group installed with different pile spacings and connected to a rigid cap are shown in Fig. 4.

Figure 4 shows that at the same loading level, the settlement of the nine-pile group connected to a rigid cap decreases with increasing pile spacing. The interactive effects among individual piles decrease with an increase in pile spacing. This is the mainly reason of the discrepancy in the behaviors of the nine-pile group with different pile spacings. For practical purpose, to effectively reduce the interactive effect among piles, the settlement of the pile groups can be decreased through increasing pile spacing properly.

For the pile groups connected to a rigid cap, the ratios of the load acted on the corner or the edge pile to the load applied on the centre pile at different loading levels are shown in Fig. 5.

Figure 5 shows that the largest, the second largest and the smallest pile loads are observed in the corner, edge, and centre piles, respectively. This is consistent with the field measured results and model test results (Cooke *et al.*, 1980; Lee and Chung, 2005). Due to the interactive effect among individual piles, the largest, the second largest and the smallest pile settlements are supposed to be presented in the corner, edge, and centre piles, respectively. However, the individual piles installed at different locations should deform in the same pace because of the restriction effect of the rigid cap installed at the pile groups. This will lead to the different loads acted on the individual piles in a group. The ratios of the load acted on the corner or edge pile to the centre pile head load gradually decrease with increasing loading level and have a uniform trend. It also can be observed that the ratios of the load acted on the corner or edge pile to the

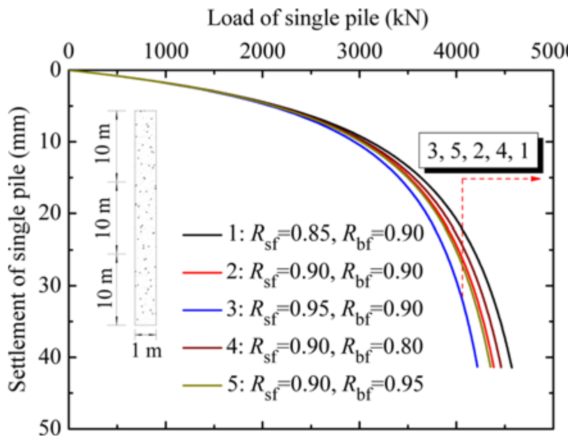

Fig. 3. Influence of R_{sf} and R_{bf} on a Single Pile Response

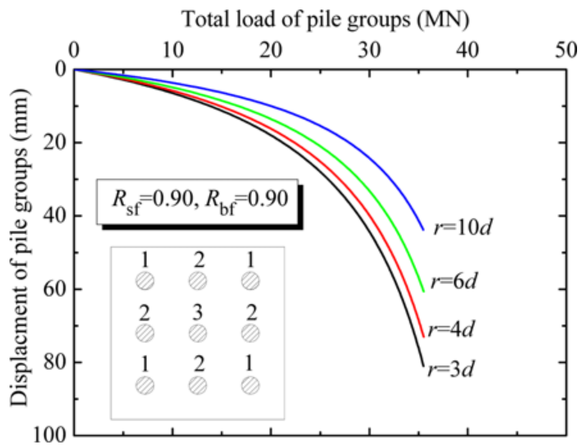

Fig. 4. Influence of Pile Spacing on Load-Settlement Responses of the Nine-Pile Group Connected to a Rigid Cap

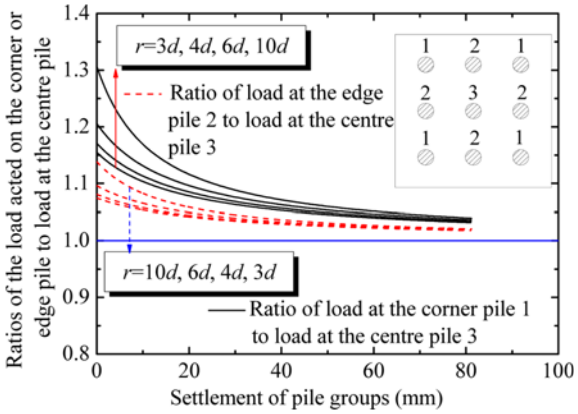

Fig. 5. Ratios of the Load Acted on the Corner or Edge Pile to the Load Applied on the Centre Pile for the Pile Groups Connected to a Rigid Cap

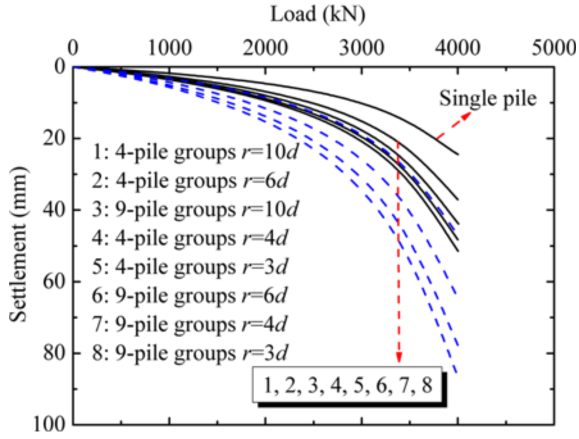

Fig. 6. Responses of Single Pile and Pile Groups at Different Loading Levels

load applied on the centre pile increase with increasing pile spacing and have a uniform trend at high loading level. This will cause the discrepancy in the interactive effect among individual piles. It is well known that the most serious interactive effect is supposed to occur in the centre pile, and the smallest interactive effect is presented in the corner pile. The discrepancy in the interactive effect of the centre, corner and edge piles increases with increasing pile spacing. This is the reason that the ratios of the load acted on the corner or edge pile to the load applied on the centre pile increases with increasing pile spacing.

3.7 Influence of Pile Spacing and Number of Piles on Pile-Group Settlement Ratio

The responses of the four-pile group and nine-pile group connected to a rigid cap and installed with different pile spacings (r = 3d, 4d, 6d and 10d, respectively) are shown in Fig. 6. In this calculation, the values of R_{sf} and R_{bf} are adopted as 0.90 for the soil around pile shaft and below the pile base, respectively.

Figure 6 shows that the settlement of the pile groups is larger than the single pile displacement at the same loading level. The settlement of the pile groups with small number of piles is larger than that of the large pile groups. This is due to the fact that the interactive effects among large pile groups are more obvious than that among the pile groups with small pile numbers.

As shown in Fig. 7, comparisons between the single pile response and the behavior of pile groups are made to capture the pile-group settlement ratio defined as the ratio of the settlement of a pile group to that of a single pile at the same average load per pile. Figure 7 shows that the pile-group settlement ratio decreases with increasing pile spacing and increases with increasing number of piles and loading level. The interactive effect decreases with increasing pile spacing, which leads to reduce the difference between the single pile behavior and the response of pile groups. This is the reason that the pile-group settlement ratio decreases with increasing pile spacing. Furthermore, the interaction among piles becomes more obvious with an increase in number of piles and loading level, which causes the pile-group settlement ratio increases with an increase in number of piles and loading level.

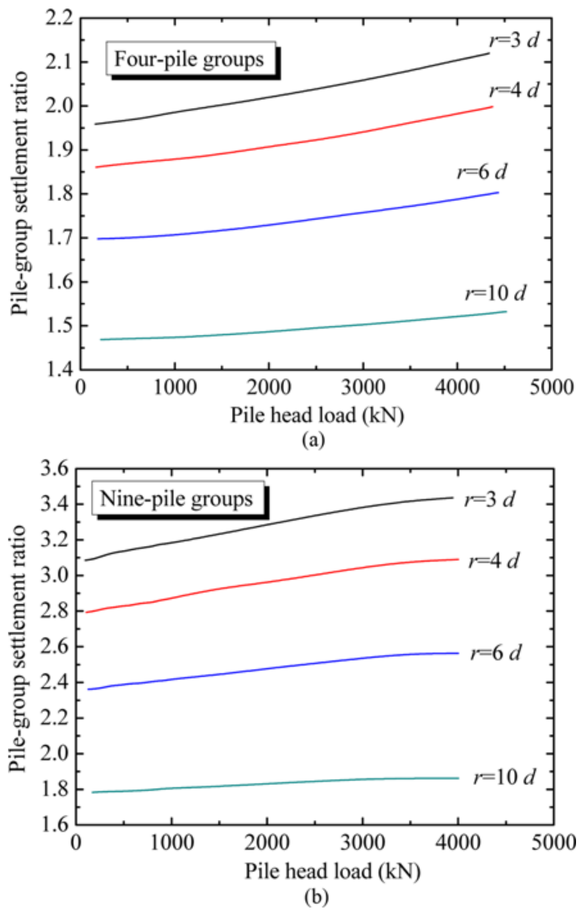

Fig. 7. Pile-Group Settlement Ratio at Different Loading Levels: (a) Four-Pile Groups, (b) Nine-Pile Groups

3.8 Influence of Pile Spacing and Number of Piles on Pile-Group Resistance Ratio

The effects of pile groups can be depicted using a pile-group

Table 1. Influence of Pile Spacing on Pile-Group Resistance Ratio

Ultimate bearing capacity of single pile (kN)	Pile spacing	Average ultimate bearing capacity of individual piles in pile groups (kN)		Corresponding settlement (mm)	Pile-group resistance ratio	
		Four-pile groups	Nine-pile groups		Four-pile groups	Nine-pile groups
4366	$r = 3d$	3772	3222	40	0.8639	0.7380
	$r = 4d$	3830	3338		0.8772	0.7645
	$r = 6d$	3925	3541		0.8990	0.8110
	$r = 10d$	4064	3860		0.9308	0.8841

resistance ratio defined as the ratio of the average resistance of an individual pile in a group to that of a single pile at the same settlement. For the single pile and the pile groups considered in this calculation, the load acted on the pile head corresponding to the pile head settlement of 40 mm is taken as the ultimate bearing capacity of the single pile and the pile groups (after the Chinese Technical Code for Testing of Building Foundation Piles (JGJ 106-2003, 2003). The ultimate bearing capacities of the single pile and the pile groups with different pile spacings and pile numbers, the pile-group resistance ratios of the four-pile group and the nine-pile group connected to a rigid cap and installed with different pile spacings are listed in Table 1.

Table 1 shows that the average ultimate bearing capacity of the individual piles in a group is smaller than the ultimate bearing capacity of the single pile due to the reduction of soil rigidity caused by the interactive effects among piles. The average ultimate bearing capacity of the individual piles increases with increasing pile spacing and decreases with an increase in the number of piles. The pile-group resistance ratio increases with increasing pile spacing. This is consistent with the field measured results (after Liu *et al.*, 1994) and the centrifuge model test results (after Xin, 2006). In practice, the bearing capacity of the pile groups can be improved by increasing pile spacing properly.

4. Conclusions

In this work, a simplified approach for nonlinear analysis of the load-displacement behavior of single pile and pile groups is presented. A hyperbolic model is used to capture the relationship between unit skin friction and pile-soil relative deformation developed along the pile-soil interface and the stress-displacement response developed at the pile end. Considering the interactive effects among piles, the values of the parameters related to the hyperbolic model of skin friction and end resistance of an individual pile in a group are obtained. As to the response of pile groups, a computational flow chart is developed. Furthermore, a parametric study is conducted to capture the influence of pile spacing and number of piles on the load-settlement response of the pile groups connected to a rigid cap. To assess the interaction effect among individual piles, the pile-group settlement ratio and the pile-group resistance ratio are analyzed. The calculation results have provided some key findings summarized as below.

1. At the same loading level, the settlement of pile groups connected to a rigid cap decreases with increasing pile spacing. For practical purpose, to effectively reduce the interactive effect among piles, the settlement of pile groups can be decreased by increasing pile spacing properly.
2. The largest, the second largest and the smallest loads are observed in the corner, edge, and centre piles, respectively. At high loading level, the loads acted on the individual piles installed at different locations trend to uniform with decreasing of pile spacing.
3. At the same loading level, the settlement of the pile groups is larger than the single pile displacement, and the settlement of pile groups with small number of piles is larger than that of the large pile groups.
4. The pile-group settlement ratio decreases with increasing pile spacing and increases with an increase in number of piles and loading level. The pile-group resistance ratio increases with increasing pile spacing. In practice, the bearing capacity of the pile groups can be improved by increasing pile spacing properly.

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