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Experimental study and numerical analysis on bearing behaviors of super-long rock-socketed bored pile groups

Gao Rui[1,2] Hu Nian[1,2] Zhu Bin[3]

([1]School of Civil Engineering, Wuhan University, Wuhan 430072, China)
([2]School of Civil Engineering, University of Nottingham, Nottingham NG7 2RD, UK)
([3]Wuhan Tianxingzhou Road-Bridge Investment and Development Co. Ltd., Wuhan 430015, China)

Abstract: A centrifuge modeling test and a three-dimensional finite element analysis (FEA) of super-long rock-socketed bored pile groups of the Tianxingzhou Bridge are proposed. Based on the similarity theory, different prototypical materials are simulated using different indicators in the centrifuge model. The silver sand, the shaft and the pile cap are simulated according to the natural density, the compressive stiffness and the bending stiffness, respectively. The finite element method (FEM) is implemented and analyzed in ANSYS, in which the stress field during the undisturbed soil stage, the boring stage, the concrete-casting stage and the curing stage are discussed in detail. Comparisons in terms of load-settlement, shaft axial force distribution and lateral friction between the numerical results and the test data are carried out to investigate the bearing behaviors of super-long rock-socketed bored pile groups under loading and unloading conditions. Results show that there is a good agreement between the centrifuge modeling tests and the FEM. In addition, the load distribution at the pile top is complicated, which is related to the stiffness of the cap, the corresponding assumptions and the analysis method. The shaft axial force first increases slightly with depth then decreases sharply, and the rate of decrease in rock is greater than that in sand and soil.

Key words: super-long rock-socketed pile; bored pile groups; centrifuge modeling test; finite element analysis

As the application of large-scale bridges and super high-rise buildings gradually increases, the bearing behavior of pile foundation is much more of concern than before. Bored piles have been widely applied in engineering practices[1-4] due to high bearing capacity and fast convergence of settlement. However, high costs and difficulties of the field tests during the experiments limit the understanding of the bearing characteristics[5].

At present, the methods of studying the pile groups mainly include the analytical method[6] based on the load-transfer function, numerical analysis methods represented by the finite element modeling (FEM)[7], the combined analytical method and finite element method[8], and the physical testing method[1-4]. The FEM is well recognized in the academic community as it can consider various factors impacting on the bearing behavior of group piles. The current finite element analysis (FEA) commonly ignores the impact of the construction phase due to multiple nonlinearity. Usually, the expected location of the wished-in-place piles model is adopted while the response during the unloading is ignored[7, 9].

Centrifuge modeling tests and the three-dimensional FEA of super-long rock-socketed bored pile groups of the Tianxingzhou Bridge have been performed. Comparisons of load-settlement, shaft axial force distribution and lateral friction between the FEM results and the test data have been carried out. Meanwhile, the bearing behaviors of super-long rock-socketed bored pile groups under loading and unloading conditions have been investigated.

1 Test Overview

1.1 Research objectives

As the bearing mechanism of piles is influenced by several factors such as the installation technology, soil behaviors, mechanical and geometrical behaviors of pile shaft and loading types, there are not adequate theory and methods to accurately predict the whole process of stress and strain mobilized in pile shafts. Consequently, the pile foundations are generally designed based on experience, and an excessive safety factor is given to ensure the structural safety. Centrifugal modeling can reflect the real stress level, which is very important to the geotechnical problems of prototypes; therefore, it is now the most effective and advanced means of reflecting the real stress level in geotechnical research. According to it, the centrifuge model test based on the prototype of the pile foundation of the Tianxingzhou Bridge is carried out with three goals:

1) To disclose the load-transfer mechanism and bearing behaviors of rock-socketed pile groups under vertical loading;

2) To investigate the influence of the ratio of length to radius;

3) To assess the safety of the pile foundation of the Tianxingzhou Bridge.

1.2 Selection of model materials

The centrifuge model test is carried out on the 400 gt geotechnical centrifuge of the Nanjing Hydraulic Research Institute (NHRI)[10], as shown in Fig. 1. With a model scale of 1∶160, a large cylindrical model box with a diameter of 800 mm and a height of 1 000 mm is used. According to the similarity theory, for sand, the natural density of the foundation plays a major role in simulation, while the compressive strength indicator of the cement conglomerate is essential for simulation of weak and moderate cement conglomerates. Similar bending stiffness is the controlling indi-

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Biography: Gao Rui (1975—), male, doctor, associate professor, gaorui@whu.edu.cn.

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cator to simulate the prototype of the pile cap. The aluminum alloy material with an elastic modulus of about 70 GPa is used to represent the concrete pile. When considering the vertical load, aluminum tubes with a diameter of 20 mm and a thickness of 3 mm are used in accordance with the shaft compressive stiffness similarity conditions. The centrifuge model is shown in Fig. 2.

Fig. 1 NHRI 400 gt geotechnical centrifuge

Fig. 2 Centrifuge test model of pile groups

1.3 Data acquisition system and loading system

The data acquisition system is composed of the pre-conditioning amplifier, the preamplifier and the computer components. The pre-conditioning amplifier is installed in the centrifuge arm end hanging near the crane platform; the preamplifier is installed near the axis of the centrifuge machine, and it is mainly used to reduce the centrifugal force of the device so that it can still work properly in the high-gravity field. During the test, the sensor laid in the model outputs the signal into the pre-conditioning amplifier and then the signal is sent to the preamplifier; after conducting A/D conversion and real-time collection, the acquired signals are uploaded to the host through the collecting circuit, the host will display, store and process the signals.

The computer-controlled electro-hydraulic servo loading system is applied. It is composed of the main shaft sleeve, actuators, load anti-aircraft force, control valves, high-precision displacement and force sensors, computer control system, all being able to meet the test requirements of arbitrary loading classifications from a single pile to group piles and then to the main tower foundation.

1.4 Procedure of centrifuge model test

The tests are carried out for 3 pile groups, namely with pile lengths of 50, 84 and 120 m, respectively. The axial force and settlement at the top of the pile cap, and the strain, stress and deformation of the pile shaft are monitored, correspondingly. The procedure can be divided into the following three steps:

1) Manufacture of the basic model element: Manufacture of the pile model and the pile cap model, and installation of fixing skeleton for piles, etc.

2) Preparation of the test: Preparation and installation of the whole model in the pile model installation, soil consolidation under designed acceleration, and installation of the pile cap and the loading system.

3) Testing of the model: The centrifuge model is accelerated to $160g$ gradually, then, loaded step by step. Meanwhile, the stress, strain, settlement and deformation are monitored.

2 Finite Element Model

At present, the FEM is the most effective numerical method for geotechnical problems. It can be used to simulate nonlinear pile-soil interactions by following a viewpoint of the overall process, and to obtain the completed distribution of stress and strain in pile shafts, soils and the platform, which is difficult for other methods. The FEA in this paper is implemented in ANSYS. A subroutine is coded by APDL which is supplied by ANSYS for user-written subroutines to establish the parametric modeling of pile groups and to analyze the overall process of bored piles from installation to unloading. Due to the significant influence on the behaviors of pile groups, the stress field during the undisturbed soil stage, the boring stage, the concrete-casting stage and the curing stage are discussed in detail, and the process of stepwise loading and unloading is analyzed. Meanwhile, convergence is a difficult point for the pile-soil-pile interaction analysis as the multi-nonlinearity of geometry, physics and boundary is involved. It can be solved by the following three measures:

1) Guaranteeing the mesh quality by rigorously controlling the distorted level of all the elements;

2) Setting reasonable values for normal and tangential stiffness and the initial penetration for the contact element;

3) Adopting a suitable nonlinear solver in ANSYS.

2.1 Mesh and boundary conditions

According to the geometric arrangement of pile groups as shown in Fig. 3, considering the symmetry of the pile layout, a quarter model is adopted in order to improve the efficiency of calculation. The length and width of the soil model are extended up to $15D$ from the cap edge (D is the pile diameter), while the depth is extended up to $0.6L$ from the pile bottom (L is the pile length). The bottom of the model is fixed and the symmetric plane is restricted symmetrically; meanwhile, the freedom of x and y directions at the lateral side is constrained as well. A vertical uniform load is distributed on the cap surface.

Concrete piles, rock and soil are all simulated by eight-node isoparametric elements (Solid45) which possess the characteristics of plastic, creep, expansion and large deformations (Release 10.0 documentation for ANSYS, ANSYS Inc, 2005). Whether or not to consider the slide and friction of the contact is extremely important for simulating the bearing characteristics of the pile foundation[11]. The surface-to-surface contact model is applied to simulate the contact behavior. The lateral surface of the pile is simulated as the target surface by TARGE170 elements, and the soil surface is simulated as contact surface by CONTAC173 elements. An improved Lagrangian algorithm is employed in the contact algorithm. For efficiency and accuracy of the calculation, meshes of interface between soil and pile are finer, as shown in Fig. 4, with 83 060 elements and 67 192 nodes.

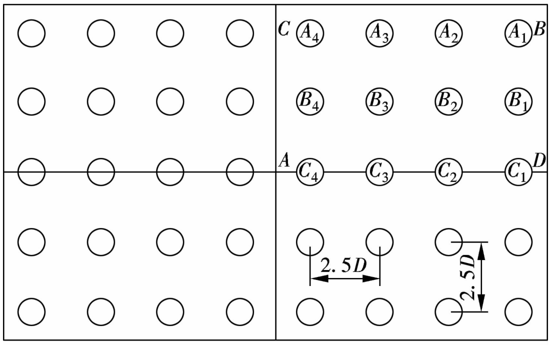

Fig. 3 Geometric configuration of pile groups (A, B, C and D are the four corner points of quarter cap)

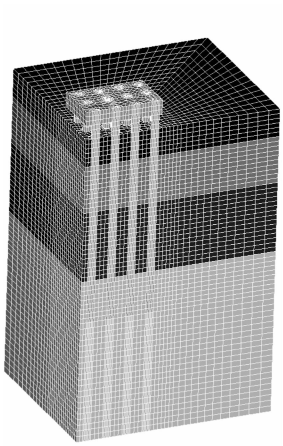

Fig. 4 FEM mesh model of pile groups

2.2 Material constitutive model and parameters

A linear elastic model is employed to simulate the pile shaft concrete, and a D-P constitutive model is employed to simulate the soil and rock (medium cement conglomerate, weak cement conglomerate and sand). In ANSYS, some other parameters such as compressive modulus E_s and internal friction angle φ should be input, as shown in Tab. 1.

Tab. 1 Material parameters used in 3D FEM analysis

Layer	Soil and rock	Thickness/m	φ/(°)	E_s/MPa
1	Sand	11	15	5.5
2	Sand	21	12	3.8
3	Weak cement conglomerate	30	22.5	150
4	Medium cement conglomerate	30	30	1 500

A Coulomb friction model is employed to predict the interface behavior between pile and soil. When $\tau < \sigma_n \tan \phi_i$, the elastic deformation will occur; when $\tau = \sigma_n \tan \phi_i$, the plastic deformation will occur. In fact, the interface friction angle ϕ_i is related to the internal friction angle φ_i of the soil material. Kulhawy[11] illustrated the ϕ/φ value of different types of pile-soil interface in detail. In ANSYS, the internal friction angle is defined as a friction coefficient $\mu_i = \tan \phi_i = \tan \varphi_i$. In addition, the maximum shear stress $\tau_{\max}$ is defined. When $\tau > \tau_{\max}$, slide occurs regardless of the value of $\sigma_n \tan \phi_i$.

2.3 Simplification of stress field

For a better simulation effect, some simplifications of the stress field are considered.

1) Simplification of the initial stress field (geostatic). Since it is difficult to measure the initial stress field, generally the geostatic stress is assumed to be equal to the gravity field. In ANSYS, the inertial acceleration g is imposed to simulate the gravity field.

2) Simplification of stress field during the boring stage. In the process of drilling, as a result of soil excavation, constraint of soil near the pile is weakened, and the horizontal effective stress is partially released. However, due to the existence of the slurry wall during boring, the stress release is very small[12]. As a matter of fact, disturbance areas are formed in the sand layer as well, which can be simulated by degrading the material parameters[13]. According to the results of the centrifuge model test, a low vertical load is shared by sand layers, and it is difficult to measure the change of the stress field accurately from the boring stage to the construction stage, so the impact of the construction phase is ignored.

3) Simplification of stress field during concrete perfusion stage. Zhu et al.[14-15] showed that the hydrostatic pressure on the borehole wall increases linearly and the increasing rate decreases in deeper areas. During the concrete perfusion stage, the concrete in the deep area behaves more like a solid rather than a liquid. The pressure on the borehole wall can be interpreted as vertical stress multiplied by the coefficient of the soil pressure, and the value should be less than the pressure of liquid concrete. Thereby, the linearly distributed pressure is applied for simplification and the lateral pressure coefficient is equal to 1.

4) Simplification of stress field during concrete curing stage. As the pile-soil interface is gradually transformed from the liquid-solid interface to the solid-solid interface, the friction angle of the contact interface is ever-increasing instead of constant. Considering the decline of the interface owing to the geotechnical remodeling during the drilling process, a small friction coefficient is assumed as the average effect in the curing stage[15-16]. Specifically, as the interface friction angle of soils is small under normal conditions, the frictional coefficients of pile-soil interfaces are conservatively assumed to be zero, while the frictional coefficients of pile-rock interfaces are assumed to be 0.1 averagely.

5) Simplification of stress field during loading stage. After the concrete curing stage, the solid-solid interface has been formed completely, therefore, during the loading stage, the normal friction coefficient can be employed.

3 Comparative Analyses

3.1 Load-settlement characteristic

Fig. 5 shows the load-settlement curves obtained by the

FEA and the centrifuge model test, which indicates that, at the loading stage, there is a good agreement between the FEA and the test results. A maximum settlement difference (4.1 mm) between the two curves can be found. The difference is less than 5%, which can definitely verify the credibility of the FEM. However, the residual settlement calculated by the FEA is significantly smaller than that by the centrifuge model test during the unloading stage. Residual settlement is mainly due to the plastic deformation of soils and the interface of the pile-soil. However, the D-P model of the soil model and the Coulomb model of the pile-soil interface cannot determine the soil behavior during the unloading and the loading stage in the FEM, which is the main cause of the difference between the FEM and the test results. Both the FEM and the test results show that an approximately linear relationship is found between the settlement and the load during the loading and unloading stages.

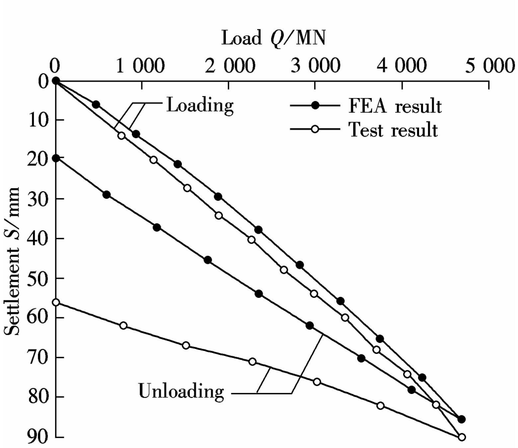

Fig. 5 Relationship between load and maximum settlement

Fig. 6 shows the load-settlement curves of bored pile caps at different locations. As can be seen from the figure, the farther from the center point A (see Fig. 3), the smaller the settlement is. In addition, during the design of the pile foundation, one needs to strictly control not only the overall settlement of caps, but also the differential settlement of caps, because the excessive differential settlement will lead to an excessive bending moment in the pile foundation. The secondary stress created in the superstructure greatly affects the construction cost. In the present study, the largest differential settlement is 23 mm, 25.5% of the largest settlement.

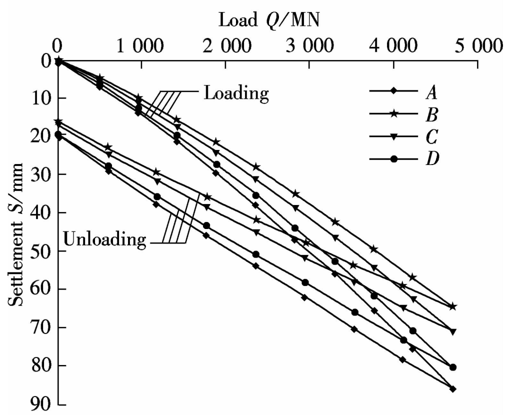

Fig. 6 Settlement in different locations (A, B, C and D are the corner points of quarter cap)

Fig. 7 shows the stress diagram of the concrete cap. We can see that the large bending moment in the cap leads to obvious bending deformation. Fig. 8 illustrates the settlement distribution on top of the cap in the loading stage. The largest settlement occurs in the central part, and the smallest one in the corners. Meanwhile, the settlement appears the same for points at the same distance from the center point.

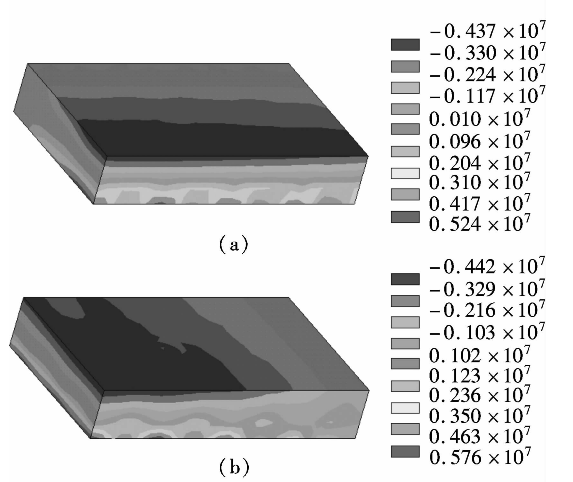

Fig. 7 Stress in the cap. (a) Horizontal stress; (b) Vertical stress

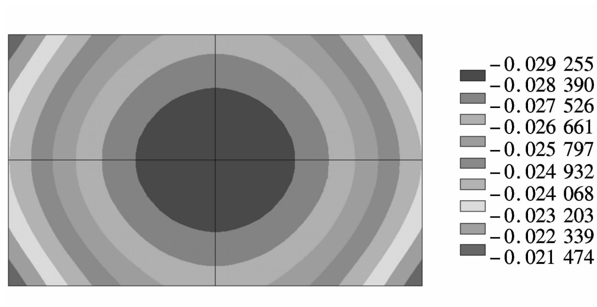

Fig. 8 Settlement distribution on the cap top

3.2 Load distribution at the pile top

Tab. 2 shows the numerical solution and test results for the load bearing ratio on the pile top of some typical locations. As can be seen from Tab. 2, different distributions can be found between the numerical solution and the test. There is a large difference between the maximum and minimum values in the test results. However, the bearing ratio is close to 1 in the numerical solution, which means that the load distribution at the pile top is even and little difference is observed.

The reasons mainly include the following aspects:

Tab. 2 Numerical solution and test results for load bearing ratio on pile top of some typical locations

Construction phase	Load/MN	Method	Pile location				
			A_1	A_4	B_3	C_1	C_4
Completion of nude tower stage	1 421	FEA	0.937	1.030	1.000	0.967	1.005
		Test	1.489	0.938	1.054	0.715	0.815
Bridge stage	2 610	FEA	0.965	1.054	0.998	0.979	1.003
		Test	1.390	0.966	1.036	0.828	0.910
Operation stage	3 330	FEA	0.968	1.060	1.010	0.980	0.981
		Test	1.319	0.960	1.038	0.866	0.952

1) In the numerical analysis, the cap is assumed to be elastic and loaded at the center; thus a concentric distribution of the settlement can be observed on the cap top (see Fig. 8). As we can see from the figure, the largest settlement is located at the center and the corresponding load of each pile top is much evener. Meanwhile, a smaller load on the corner pile and a larger load on the central pile can be observed.

2) The test results are not of obvious regularity, which is related to the discreteness of the equipment and the test data.

3.3 Distribution of shaft axial force

Fig. 9 illustrates the shaft axial force distribution of the super-long rock-socketed bored piles. It reveals that the same distribution law of shaft force can be found between the FEA and the centrifuge model test as follows:

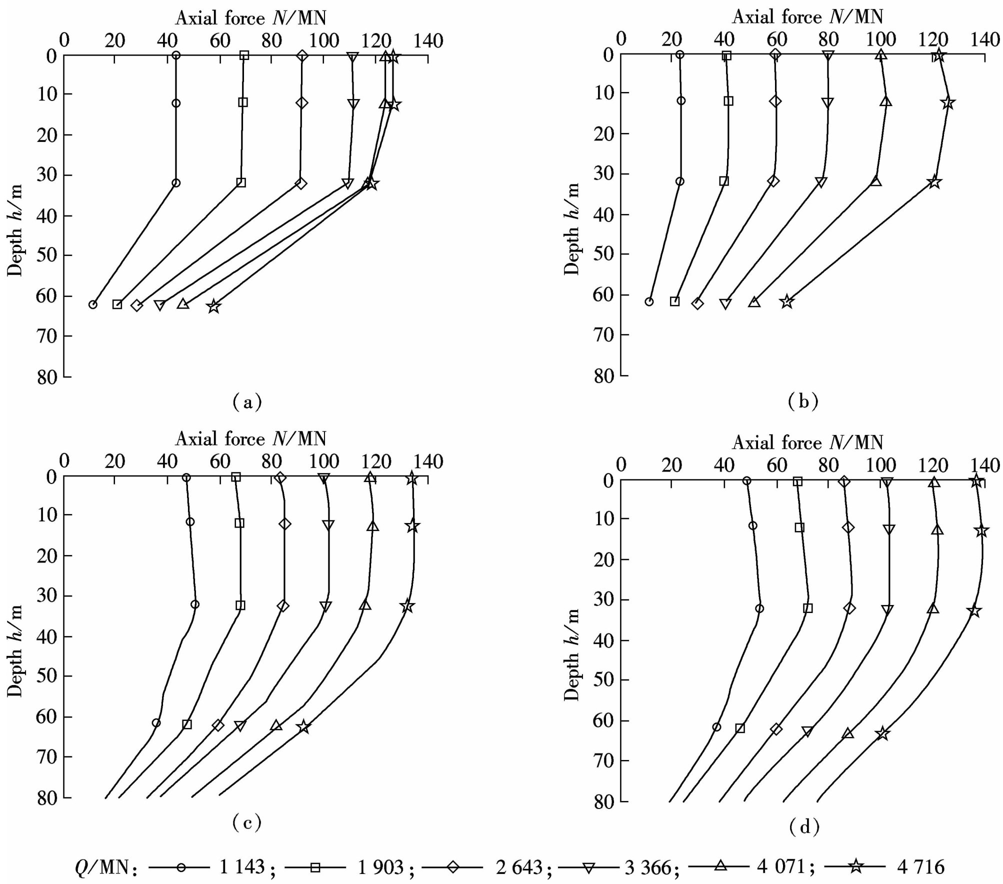

Fig. 9 Shaft axial force distribution. (a) Test result of pile A_1; (b) Test result of pile C_4; (c) FEA result of pile A_1; (d) FEA result of pile C_4

1) The shaft axial force first increases slightly with depth, then decreases slowly with little change, while in the rock layer, the shaft axial force decreases dramatically. According to Ref. [17], soil layering has a complicated influence on the pile-soil-pile interaction, which not only induces an additional settlement to individual pile in groups but also induces an additional stress along its shaft. As for a single pile, the stress along the shaft decreases with depth, while the additional stress increases with depth; therefore, there exists a point where the axial force reaches maximum value along the pile shaft.

2) At the interface between two soil layers, there is an obvious turning point in the distribution curve. In contrast to the fractional stress along the pile shaft, it can be seen that there is a discontinuity at the interface of two soil layers as the soil physical parameters change suddenly (see Fig. 10), which is the reason why the axial force changes abruptly at the interface.

3) The shaft axial force increases as the load increases; however, the change of the shaft axial force with the depth in the area close to the pile top is not obvious under the same load level.

4) The axial force distribution laws are similar in the piles at different locations.

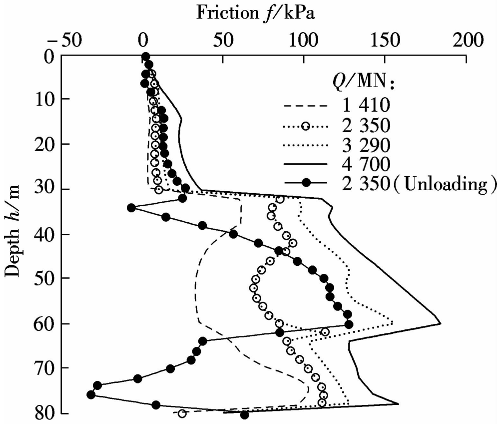

Fig. 10 Shaft axial force distribution of pile C_4

4 Conclusions

1) A finite element model is developed and validated by the centrifuge model test. Good agreement can be found between the FEM and the test results.

2) The load-settlement curves show an approximately linear relationship during the loading and unloading stages.

3) Complicated load distribution at the pile top can be observed, which is related to the stiffness of the cap, the corresponding assumptions and the analysis method.

4) Overall, the shaft axial force decreases with depth. The decreasing rate in rock is greater than that in sand and soil, which is similar to the bearing behavior of point bearing friction piles.

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超长钻孔灌注群桩承载特性的实验研究和数值分析

高 睿[1,2] 胡 念[1,2] 朱 斌[3]

([1] 武汉大学土木建筑工程学院,武汉 430072)

([2]School of Civil Engineering, University of Nottingham, Nottingham NG7 2RD,UK)

([3] 武汉天兴洲道桥投资开发有限公司,武汉 430015)

摘要:采用离心机模型试验和三维有限元模型对天兴洲大桥的超长嵌岩钻孔灌注群桩进行了分析. 离心机试验中,根据相似理论,不同的原型材料用不同的指标进行模拟,细砂、桩身、承台分别以天然密度、抗压刚度、抗弯刚度作为指标进行模拟. 对土体未开挖阶段、钻孔阶段、混凝土灌注阶段和养护阶段的应力场进行了详细的讨论,并应用 ANSYS 进行了分析. 根据 2 种方法得到的结果,从荷载沉降曲线、桩身轴力分布和侧向摩阻分布等方面,对比分析了加载和卸载条件下超长钻孔群桩的承载特性. 结果表明,有限元计算模型与离心机试验模型沉降观测的结果吻合得很好;桩顶反力的分布规律复杂,与承台的自身刚度、相应的假定和分析方法有关;轴力随着深度先稍微增加,后逐渐减少,并且在岩层中减少的速度远比砂土中快.

关键词:超长嵌岩群桩;钻孔灌注群桩;离心机模型试验;有限元分析

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