Research Article

Experimental Study on Static Load of Large-Diameter Piles in Nonuniform Gravel Soil

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With the development of tourism, the number of multistorey buildings in mountain areas is increasing gradually, and the requirements of the form and bearing capacity of foundation in landslide areas are getting more demanding than ever. In-situ testing of rock and soil mass in slope area has important practical significance for improving the stability of building foundation. Taking a project in Baishi Mountain located in southwest of China as an example, firstly, the geological structure and mechanical properties of soil are analyzed. Then, two types of pile foundations, i.e., empty-bottom pile foundations and solid-bottom pile foundations, are designed based on the characteristics of the geological structure for carrying out the static load test on pile foundation. The test results are as follows: (a) the load settlement curve (Q-S) of the empty-bottom test pile shows a steep drop, while the Q-S curve of the solid-bottom test pile shows a gradual change, showing that the end-bearing friction pile's property and the ultimate bearing capacity of the solid-bottom pile are higher than those of the empty-bottom pile. (b) The maximum lateral friction of the four test piles is 139.158 kPa, 148.015 kPa, 150.828 kPa, and 154.956 kPa, respectively. (c) The shaft skin resistance under ultimate load is coming close to the maximum value, and the maximum values are 9.792 mm, 7.939 mm, 9.881 mm, and 14.97 mm, respectively. Research results can serve as design bases for the pile foundation of multistorey buildings located in landslide areas of Baishi Mountain in the southwest of China and also as references for the engineering application of pile foundation in similar geological fracture areas.

1. Introduction

Due to its high bearing capacity, small settlement, good stability, and other characteristics, pile foundation has been extensively studied by many scholars around the world in the past few decades. In the early research of pile foundation, the integrity of pile body and the bearing capacity of single pile are mainly tested by low strain integrity testing [1, 2], acoustic transmission method [3], penetration test (CPT) [4], and other methods, but the bearing capacity of pile foundation obtained by the above methods is only estimated value.

The field static load test method is the most reliable method to determine the bearing capacity. In recent years,

according to the field static load test research of bored piles with different length-diameter ratios, it has been confirmed that the bearing performance of the piles is related to the length-diameter ratio of the piles and the construction technology of the pile foundation itself; increasing the bottom of pile and no sediment at the bottom will improve the bearing capacity of pile, and the larger the length-diameter ratio, the lower the bearing capacity [5–7]. In addition to the specific length-diameter ratio, the rock-soil properties [8] where the pile foundation is located also affect the bearing capacity. Therefore, in the foundation design, the effective stress distribution of the soil around and below the foundation unit must be determined first, because it is the basis for

design analysis [9]. Until now, scholars around the world have also carried out extensive load test research on the bearing capacity of pile foundation in different rock and soil mass, including laboratory tests [10, 11] and field static load tests of pile foundation in different geologic structure in different regions. Guo and Ghee [12] studied the influence of lateral movement of upper soil mass on pile bending moment and soil reaction force through model test and found that when there is vertical load on the pile top, the maximum deformation of the pile body, the maximum bending moment, and the soil reaction caused by the movement of the upper soil mass are obviously smaller than those caused by no vertical load on the pile top. According to Zhou et al. [13], taking Wuding Expressway project in loess region as an example, the shaft skin resistance of six test piles is studied through static load testing and multiparameter statistical analysis. The multiparameter statistical analysis method is compared with the static load test results, and the error is controlled within 20%. Hai and Fellenius [14] conducted static loading tests using single-level O-cells at Ho Chi Minh City, Vietnam, and showed the shaft resistance to be postpeak softening, and the pile toe stress-movement responses were essentially linear and almost identical for the two piles. With the development of the computer numerical simulation technology such as finite element method and finite difference method, the numerical simulation method is gradually applied to the prediction and evaluation of the bearing capacity of pile foundation [15, 16].

The numerical simulation calculation results and the experimental results are easily affected by soil properties, and there are few researches on the bearing capacity of pile foundation in the landslide area. Therefore, in this paper, we study the field in which the vertical static load test of large diameter manually bored cast-in-place pile was carried out in the Baishi Mountain located in southwest of China, to study the bearing performance and load transfer law of pile foundation in the slope area.

2. Soil Conditions

The test is carried out in the slope area of Baishi Mountain. The rocks in the test site are in different sizes and shapes, and the geologic structure is uneven. Before the static load test of pile foundation, multichannel transient surface wave test and superheavy (N_{120}) cone dynamic sounding test are conducted on the test site area in order to determine the physical and mechanical properties of gravel soil.

In the multichannel transient surface wave test, DZQ24-2A series high-resolution seismograph and 24-pound hammering source are used. There are 10 observation systems with a distance of 2 m between each two. The minimum offset distance is 10 meters, the sampling interval is $512 \mu s$, and the total number of sampling points is 1024. In view of the difference in frequency and propagation speed between surface wave and other seismic wave fields and the dispersion characteristic of surface wave, $f-k$ analysis method [17] was used in data processing, and the shear wave velocity profile was generated after Kriging meshing and Gaussian smoothing. As shown in Figure 1, the abscissa is the section length L (m) and the ordinate is the formation depth Z (m).

FIGURE 1: Profile of shear wave velocity on site area.

According to the profile of shear wave velocity in Figure 1, the site can be divided into four layers. The first layer is cultivated soil with a depth of about 1 meter, and an average shear wave velocity is less than 150 m/s. The second layer is sandy soil, a depth about 1 to 3 meters, and an average shear wave velocity of 150 to 200 m/s. The third layer is silty clay with a relatively a small amount of gravel soil content, a depth about 3 to 6 meters, and an average shear wave velocity of 200 to 250 m/s. The fourth layer is gravel soil with a depth of more than 6 m and a wave velocity greater than 250 m/s. In the longitudinal direction, the wave velocities of surface soil and deep soil are quite different. In the transverse direction, the distribution of chromatic aberration in shear wave velocity image is chaotic. According to the above findings, it can be concluded that the shear wave velocity of rock and soil mass on the site changes greatly and the testing site is composed of many kinds of rock and soil with different sizes and shapes and shows the characteristics of strong nonuniformity.

Cone dynamic sounding test is a commonly used in-situ test method to evaluate the mechanical properties of granular soil. In this test, 4 locations are selected as test points and superheavy 120 kg dynamic penetrometer was used. The statistical results of penetration hammering counts are shown in Table 1. The relationship between hammering counts and penetration depth is shown in Figure 2.

According to the standard values of the superheavy N_{120} dynamic sounding hammering count of the four test points and China Code for Investigation of Geotechnical Engineering (GB50021 2016) [18], it can be concluded that the density of gravel soil in the site is of medium density. From the single-hole hammering count obtained from the test, it can be seen that high hammering counts and low hammering counts are interwoven, indicating that the particle size of rock and soil mass on the site is complicated and the uniformity is poor. In order to investigate the grain size relationship of soil and rock of each test pile, the field stratified sampling and screening test was carried out on the gravel soil. Due to the crushing of machines and tools, the topsoil (about 1 m thick) was removed before the test. Samples were taken from the point of 1 m and then sampled at intervals of 3 m in sections, with each section sampling about 200 kg. After sampling, the soil was dried in an oven ($105\sim110^{\circ}C$), and the block stones with a short side diameter of more than 60 mm were weighed. Then, the remaining

TABLE 1: Statistical table of test indexes of superheavy dynamic sounding test (N_{120}).

Test point	Scope of statistics (m)	Range value	Average hammering counts for single-hole correction N_{120}	Standard value	Variable coefficient
No. 1	0.3~10.0	0.96~19.552	7.55	4.60	0.609
No. 2	0.3~10.0	1.82~19.552	8.82	4.36	0.49
No. 3	0.3~10.0	1.88~17.766	8.56	3.86	0.450
No. 4	0.3~15.0	1~20.032	7.88	3.98	0.505

FIGURE 2: SPT N_{120} values with depth.

soil was placed on the standard screen selected in batches of 10 kg each time for vibration screening for 10 to 15 minutes. Finally, the mass of samples on each screen was weighed and the soil gradation was calculated. The statistical analysis of test data is shown in Table 2. And the grain size distribution curves at the site of A, B, C, and D are shown in Figure 3.

During on-site screening, at positions A, B, and C, with a depth from 1 to 4 m, the particle sizes of rock and soil bodies differ greatly, with missing intermediate particles, faults, nonuniform soil particles, and easy compaction. At D screening positions, the particles with a depth of less than 4 m are relatively uniform and easy to compact. According to statistical data, the nonuniformity coefficient C_u of gravel soil is 4.042 to 35.641; the average value is 9.461. The curvature coefficient C_c is 2.45 to 4.359, with an average value of 3.227. According to the grain size distribution curves (as seen in Figure 3), it can be seen that, on the site, there is a large difference in particle size, soil particles are nonuniform, and particles are loose. The site is a poorly graded site and is not conducive for stabilizing the foundation.

According to the results of the above-mentioned multichannel transient surface wave test, superheavy (N_{120}) cone dynamic sounding test, and gravel soil particle grading test, it is concluded that the geological structure of the site is complex, the content of gravel soil is high, and the non-uniformity between particles is high. Based on the preliminary engineering survey report and the above-mentioned in-situ test results, detailed physical parameters of the rock and earth mass on the site are obtained, as shown in Table 3.

TABLE 2: Statistics and analysis of gravel soil screening test and conventional physical parameters.

Specimen number	Sample depth (m)	C_u	C_c
A	1~4	15.643	4.357
A	4~7	4.042	2.494
A	7~10	6.228	3.232
B	1~4	17.627	3.303
B	4~7	5.297	3.175
B	7~10	4.66	2.763
C	1~4	35.641	4.359
C	4~7	7.738	3.806
C	7~10	5.333	2.91
D	1~4	9.133	2.737
D	4~7	4.67	2.747
D	7~10	6.853	4.123
D	10~13	5.48	2.72
D	13~15	4.105	2.45
Statistics		14	14
Average value		9.461	3.227
Maximum value		35.641	4.359
Minimum value		4.042	2.45

3. Pile Test Program

3.1. Pile Foundation Construction. According to the above-mentioned in-situ test results, in order to study the internal force characteristics of pile shaft axial force, shaft skin resistance, ultimate tip resistance, and other internal force

FIGURE 3: The grain size distribution curves at the site of A, B, C, and D.

TABLE 3: Physical and mechanical properties of soil.

Soil layer	Weight density γ (kN/m ³)	Internal friction angle φ (°)	Cohesion C (kPa)
Cultivated soil	18	18	15
Sandy soil	19	21	—
Silty clay	19	13	23
Gravel soil	20	25	5

characteristics of pile foundation in gravel soil bearing stratum of different depths, static load tests are carried out in the area where shear wave velocity is relatively uniform, i.e., within 30 to 38 m along the length direction of wave velocity profile. The test pile is a C35 manually excavated cast-in-place pile with a diameter of 0.9 m. A total of 4 piles were arranged. See Figures 1 and 4 for the plan layout and pile numbers. The characteristic parameters of each tested pile are shown in Table 4.

The No. 1 pile is empty-bottom pile (as shown in Figure 5(a)) and Nos. 2, 3, and 4 piles are solid-bottom piles (as shown in Figures 5(b) and 5(c), resp.). The empty-bottom pile is welded to a 20 mm thick circular steel plate at the bottom of the steel cage; the diameter of the steel plate is (850 mm) slightly smaller than the diameter of the pile hole. When pouring concrete, the steel cage is suspended to make the bottom of the steel cage suspended 1 m.

There are 12 Φ 14 HRB 400 reinforcements used as main reinforcements of all test piles. The inner stirrup adopts HPB235 reinforcement with a diameter of 16 mm, and it starts from 2 m below the top of the longitudinal reinforcement and is evenly arranged each 2 m. The outer stirrup adopts HPB235 reinforcement with a diameter of 8 mm, and it is arranged from below the longitudinal rib top. The spacing between the outer stirrup and the longitudinal rib top is 100 mm within the range of 3 m, and the rest of the space is 200 mm. At the same time, 6 anchor piles with a length of 15 m are arranged, in 4 of them the diameter is 1300 mm, and the others' diameter is 1100 mm. The

FIGURE 4: Layout of pile position.

longitudinal bar of anchor piles is composed of 16 and 20 finished reinforcing bars with the diameter of 28 mm. The concrete reinforcement layout and strain gauges layout of the test pile are seen in Figure 5.

3.2. Measuring Instrument. Resistive strain gauges are symmetrically arranged on the main reinforcement (as shown in Figure 5) to measure the vertical strain of reinforcement and pile body. The DM-YB1840 dynamic and static strain testing system automatically collects strain data and calculates axial force on pile body and shaft skin

TABLE 4: The characteristic parameters of each tested pile.

Piles' no.	Piles' length L (m)	Piles' diameter D (mm)	Concrete strength	Form
No. 1	10	900	C35	Empty-bottom
No. 2	10	900	C35	Solid-bottom
No. 3	10	900	C35	Solid-bottom
No. 4	15	900	C35	Solid-bottom

FIGURE 5: The reinforcement layout and strain gauges layout of the test pile. (a) Empty-bottom pile No. 1. (b) Solid-bottom pile No. 2 and No. 3. (c) Solid-bottom pile No. 4. (d) Plan of layout of pile.

resistance. The embedment of strain gauges shall be carried out strictly in accordance with the operating instructions and maintenance work shall be done to improve the success rate; for the on-site embedding photo of strain gauges, see Figure 6.

3.3. Static Load Method of Pile Foundation. In the YD10000-200 hydraulic jack applying load, the load reaction force is provided by anchor pile. Four FP-50 displacement sensors were used for settlement, which were installed on the reference beam at the top of the test pile. The support of the sensor was fixed on the jack with magnetic bearing, and the reference beam was fixed on the steel beam in an independent position unaffected by deformation calibrating the displacement sensor with the displacement sensor calibration table before the test and using a force transducer to calibrate the accuracy of the load on the jack. The Y-LINK static load test system automatically collects and stores test data. The vertical compressive static load device of single pile is shown in Figure 7(a), and the photo of field static load test is shown in Figure 7(b).

The test is carried out according to China Technical Code for Testing of Building Foundation Piles (JGJ106-2014) [19]; the specific steps are as follows.

(1) In the test, the step loading method was adopted; (2) measuring the settlement every 5 minutes, 10 minutes, and 15 minutes after each step of loading was applied, every 15 minutes after that, and every 30 minutes after a total of 1 hour and then loading stopped. (3) The settlement per hour could not exceed 0.1 mm. When it occurred twice in succession, it was considered to be relatively stable and the next step of loading could be applied. (4) When the settlement of the pile was 5 times of the settlement under the previous load and the total settlement at the top of the pile exceeded 40 mm, or the settlement of the pile was greater than 2 times of the settlement under the previous load, and had not reached a stable state after 24 hours, loading stopped.

3.4. Test Results of Pile Foundation. The line type of load-displacement (Q-S) curve in static load test can macroscopically reflect pile body's failure mechanism and failure mode of soil around pile [20]. When the load is small, the displacement curve tends to develop linearly and is in the elastic stage. With the increasing load, the pile gradually changes from elastic deformation to elastic-plastic deformation. As the load continues to increase, the displacement increases sharply and enters the plastic stage and the pile reaches the failure state. The Q-S curves of piles Nos. 1, 2, 3, and 4 are shown in Figure 8.

It can be seen from Figure 8 that the Q-S curve of No. 1 pile has obvious inflection point, indicating that it is a steep drop curve. The pile top load corresponding to the inflection point of Q-S curve is taken as the ultimate bearing capacity of the test pile [21]. The Q-S curves of the other three piles have similar trends, and there is no obvious steep drop section, indicating that they are slowly changing curves, so the ultimate bearing capacity is confirmed according to 4.4.2 in *The Technical Specification of China Building Foundation*

Piles. The bearing capacity under ultimate load of No. 1 pile is 3600 kN, and the corresponding pile top settlement is 12.89 mm; the maximum load is 4000 kN and the maximum settlement is 40.68 mm; similarly, the maximum loading load of Nos. 2, 3, and 4 piles and their corresponding pile top displacement values, bearing capacity under ultimate load and corresponding pile top displacement, and settlement values are shown in Table 5.

Figure 9 shows the S-lgt curve of settlement of each pile with time, which can further explain the value of ultimate bearing capacity. At the last step of loading, an obvious downward bending is shown in the displacement of the four piles, the displacement exceeded 40 mm, and the pile reaches the failure state [22].

4. Discussion

4.1. The Characteristics of Pile Shaft Force Transfer. The reinforcement strain $\varepsilon_i(\mu\epsilon)$ at the corresponding depth is directly measured by the strain gauges meter embedded, and then the reinforcement stress σ_i is calculated according to

$$\sigma_{si} = K \cdot (\varepsilon_0 - \varepsilon_i), \quad (1)$$

where σ_i is the stress on the reinforcement; K is calibration coefficient indicated by the manufacturer and its value ranges from 0.6 to 0.8; ε_i is average strain measured by strain gauges on the same section; i is the serial numbers of the buried sections of the strain gauges, $i = 1, 2, L, n$ are arranged from top to bottom.

According to Hooke's law and the stress on the reinforcement, the real strain value of the strain gauges string is calculated:

$$\varepsilon_{si} = \frac{\sigma_{si}}{E_s}, \quad (2)$$

where ε_{si} is the real strain of the strain gauges string; E_s is the elastic modulus of reinforcement, with a value of 2.04×10^4 MPa; σ_i and i are the same as in formula (1).

Assuming that the compression amount of concrete is equal to the real strain value of the strain gauges string and the cross-sectional area of the pile is unchanged, the concrete stress can be obtained according to

$$\sigma_{ci} = \varepsilon_{si} \cdot E_c, \quad (3)$$

where σ_{ci} is the concrete stress value; ε_{si} is the real strain of strain gauges string; E_c is the modulus of elasticity of concrete, with a value of 10^4 MPa; i is the same as in formula (1).

Finally, the respective internal forces are obtained by multiplying stress values by the respective cross-sectional areas, and then the pile axial force of this section is obtained by the stress of reinforcements plus the stress of concrete, and the axial force of other cross-sectional pile bodies can be obtained similarly; the calculation formula is as follows:

$$Q_i = \sigma_{ci} A_{ci} + \sigma_{si} A_{si}, \quad (4)$$

where Q_i is the axial force of tested section; σ_{ci} and σ_{si} are the same as in formulas (3) and (1), respectively. A_{ci} is the

FIGURE 6: Site photo of strain gauges embedding. (a) Part photo. (b) The overall photo.

FIGURE 7: Schematic diagram and field photo of static load test. (a) Schematic diagram. (b) Photograph.

FIGURE 8: Load-displacement curve of static load test.

concrete area of pile body, 0.63585 m^2 ; A_{si} is the area of reinforcement in pile body, $1.84632 \times 10^{-3} \text{ m}^2$.

Figures 9(a)–9(d) show the axial force distribution of piles Nos. 1, 2, 3, and 4 with depth under different loading steps,

respectively. With the increase of loading steps, axial force is gradually generated at the lower part of each solid-bottom pile, and pile tip resistance is gradually mobilized [23]. When reaching the ultimate load, pile tip resistance of piles Nos. 2, 3, and 4 is 2100 kN, 1418 kN, and 3463 kN, respectively. The first pile is an empty-bottom pile; with the increase of load, axial force is gradually generated at the lower part of the pile body. After reaching the ultimate bearing capacity, the load will continue to be applied, and the displacement of pile top will rapidly increase from the original 12.89 mm to 40.68 mm. This is because the first pile is an empty-bottom pile, and there is no soil at the bottom to generate tip resistance. When the shaft skin resistance reaches the maximum value and cannot resist the axial force in the vertical direction, if the load continues to be applied after reaching the ultimate bearing capacity, sudden settlement of the pile body occurs and the settlement exceeds the limit specified in the specification, indicating that pile failure has occurred.

It can be seen from the stress conditions of the solid-bottom pile, the empty-bottom pile, the 10-meter-long pile body, and the 15-meter-long pile body in Figure 10 that the axial force of the empty-bottom pile and the solid-bottom pile is relatively small at the initial loading step. With the increase of loading step, the axial force gradually develops

TABLE 5: Results of static load test.

Pile number	Maximum load (kN)	Maximum load settlement (mm)	Bearing capacity under ultimate load (kN)	Settlement under ultimate load (mm)	Rebound rate (%)
No. 1	4000	40.68	3600	12.89	10.06
No. 2	5400	40.1	4800	30.01	12.36
No. 3	4800	40.51	4200	28.92	16.84
No. 4	8800	40.83	8000	26.76	18.94

FIGURE 9: S-lgt curve of each pile. (a) No. 1 pile. (b) No. 2 pile. (c) No. 3 pile. (d) No. 4 pile.

FIGURE 10: Distribution curve of axial force of each pile. (a) No. 1 pile. (b) No. 2 pile. (c) No. 3 pile. (d) No. 4 pile.

and transmits to the pile bottom and decreases with the increase of the depth. Under different loading steps, the attenuation rate of axial force curve is different. The attenuation rate of axial force curve reflects the development of shaft skin resistance. In low loading steps, the attenuation rate of axial force curve is not fast and relatively uniform. With the increase of loading steps, the attenuation rate of axial force curve at the upper part of pile body is not fast, while the attenuation rate of axial force curve at the lower part of pile body is obvious, which shows that no matter it is an empty-bottom pile or solid-bottom pile, the force

transmission characteristics are similar. With the increase of loading steps, the axial force gradually transmits to the lower part of the pile body. If load continues to be applied after reaching the ultimate load, the empty-bottom pile will be suddenly destroyed while the solid-bottom pile will be able to bear due to soil in the lower part and will show a slow change.

4.2. Analysis of Characteristics of Shaft Skin Resistance of Pile Body. Based on the principle of static balance, regardless of the influence of pile body weight [24], the pile shaft skin

FIGURE 11: Distribution of shaft skin resistance on each pile. (a) No. 1 pile. (b) No. 2 pile. (c) No. 3 pile. (d) No. 4 pile.

TABLE 6: Value of shaft skin resistance of piles at different depths when reaching ultimate bearing capacity.

(a) No. 1 pile									
Depth Z/m	1.7	3.4	5.1	6.8	8.5				
Skin friction/kPa	60.177	112.974	133.037	139.158	135.235				
(b) No. 2 pile									
Depth Z/m	1.6	3.2	4.8	6.4	8.1	9.8			
Skin friction/kPa	43.899	105.055	128.283	139.1	146.983	148.015			
(c) No. 3 pile									
Depth Z/m	1.6	3.2	4.8	6.4	8.1	9.8			
Skin friction/kPa	46.948	106.907	130.723	145.894	147.938	150.828			
(d) No. 4 pile									
Depth Z/m	1.6	3.2	4.8	6.4	8	9.6	11.2	13	14.8
Skin friction/kPa	63.055	93.009	117.592	129.804	141.642	144.953	145.865	151.216	154.956

FIGURE 12: Pile-soil relative displacement curve. (a) No. 1 pile. (b) No. 2 pile. (c) No. 3 pile. (d) No. 4 pile.

resistance value can be calculated by the following formula according to the change value of the axial force of the pile body:

$$q_{si} = \frac{Q_i - Q_{i+1}}{u \cdot l_i}, \quad (5)$$

where q_{si} is the resistance between the i section and the $i + 1$ section; i is the same as in formula (1); u is the perimeter of pile body; l_i is the pile length between the i section and the $i + 1$ section.

Figures 10(a)–10(d) show the distribution of shaft skin resistance of piles Nos. 1, 2, 3, and 4 with depth under different steps of loading.

As can be seen from Figure 11, in lower loading step, the shaft skin resistance of each pile is small; with the increase of loading steps, the shaft skin resistance of each pile increases. On the whole, the shaft skin resistance of the pile is also gradually transmitted from the upper part of the pile body to the lower part of the pile body, but, compared with the middle part of the pile body, the shaft skin resistance of the lower part of the first pile body with empty-bottom shows a

FIGURE 13: Distribution curves of shaft skin resistances versus relative displacements. (a) No. 1 pile. (b) No. 2 pile. (c) No. 3 pile. (d) No. 4 pile.

decreasing trend, because there is no soil bearing the corresponding load at the pile tip; however, for the remaining three solid-bottom piles, when the tip resistance is mobilized to a certain extent, the friction resistance in the middle part of the pile body is smaller than that in the lower part, which indicates that the soil at the pile tip can reinforce the shaft skin resistance of the pile [25]. When the ultimate bearing capacity is reached, the shaft skin resistance values of the corresponding piles at various depths are shown in Table 6.

4.3. Relationship between Shaft Skin Resistance and Pile-Soil Relative Displacement. The pile-soil relative displacement objectively reflects the mobilization of shaft skin resistance.

According to the calculation method in [26], the pile-soil relative displacement curves under different loading steps are calculated and drawn. The curves of shaft skin resistance and pile-soil relative displacement are shown in Figures 12(a)–12(d), respectively.

In the calculation of the pile-soil relative displacement, it is assumed that the soil around the pile will be not displaced. As can be seen from Figure 12, the pile-soil relative displacement at the same position of the pile body increases with the increase of loading steps on the pile tip. When the first two steps of loading are applied, the pile-soil relative displacement of the four piles increases obviously due to the compaction effect of the first two steps of loading on the gravel soil around the pile. As the load level continues to increase, the pile-soil

relative displacement increment is relatively small because the shaft skin resistance gradually mobilizes its function to prevent the pile from moving. With the increase of loading, when the shaft skin resistance cannot resist the load, at the loading sections 4800 kN–5400 kN, 4200 kN–4800 kN, 8000 kN–8800 kN, and 3600 kN–4000 kN, the maximum displacements of pile tips of Nos. 2, 3, and 4 solid-bottom pile and the empty-bottom pile increase by 9.792 mm, 7.939 mm, 9.881 mm, and 14.97 mm, respectively.

The shaft skin resistance increases with the increase of pile-soil relative displacement. Although continuous loading has led to foundation soil destruction, it can be concluded from Figure 13 that the lateral resistance in the middle and lower parts is not fully mobilized, and a certain pile-soil relative displacement is needed to promote the full mobilization of the shaft skin resistance. However, according to the development trend that the slope of the shaft skin resistance gradually decreases, it can be inferred that the shaft skin resistance will reach the maximum value. At the first section position, the ultimate shaft skin resistance of piles Nos. 1, 2, 3, and 4 are 46.95 kPa, 43.90 kPa, 60.18 kPa, and 63.05 kPa respectively, and the corresponding pile-soil relative displacements are 34.19 mm, 34.42 mm, 26.33 mm, and 32.79 mm. In the second section, the shaft skin resistance of the four piles increases greatly, so does the pile-soil relative displacement. Since the soil above the first section position is basically cultivated soil, and it can resist small shaft skin resistance, while the content of gravel soil of the second section is large, and, with the increase of depth, the surrounding pressure of soil around the pile increases, therefore the ability of resisting pile foundation sliding and displacement increases. Therefore, between the first section and the second section, the shaft skin resistance increases greatly, while the pile-soil relative displacement increases a little. In the second section and beyond, the soil is gravel soil, and the shaft skin resistance increases slightly with the increase of pile depth, and the relative displacement decreases gradually.

4.4. Pile Tip Resistance Varies with Load and Displacement. During the test, the section where the largest embedded depth of strain gauges is very close to the bottom of the pile was shown, so the resistance at the pile tip is approximately equal to the axial force measured at this section, namely, $Q_b = Q_0$. Under the action of all loading steps, the resistance variation curves at the lower tips of the Nos. 2, 3, and 4 piles are shown in Figure 14.

The No. 2 and No. 3 piles in the first two loading steps and No. 4 pile in the first loading step have smaller resistance at the pile tip. The resistance at the pile tip gradually increases until the third and fourth loading steps. Under the action of ultimate load, the ratio of pile tip resistance to pile top load of Nos. 2, 3, and 4 is 37.7%, 30.08%, and 39.92%, respectively, all higher 15%, which shows that pile tip resistance has been brought into play to a certain extent. The pile tip resistance is shared by shaft skin resistance and pile tip resistance, and the pile is of tip bearing friction pile.

According to the relationship curve between pile tip resistance and pile tip displacement shown in Figure 15, it

FIGURE 14: Variation curve of pile tip resistance under different loading steps. (a) No. 1 pile. (b) No. 2 pile. (c) No. 3 pile. (d) No. 4 pile.

FIGURE 15: Relationship curve between pile tip resistance and pile tip displacement. (a) No. 1 pile. (b) No. 2 pile. (c) No. 3 pile. (d) No. 4 pile.

can be seen that the pile tip resistance and pile tip displacement present a double fold line model of work hardening. The Nos. 2, 3, and 4 piles all show a slow deformation failure, and, under the action of the first loading step, the slope of the curve is larger, and the corresponding displacement is relatively small; in latter steps of loading, the curvature curve becomes smoother, the displacement of the pile tips of adjacent loads increases, and the soil at the pile tips will gradually be compacted.

5. Conclusions

Analyses are carried out on data of the in-situ test of uneven gravel soil in Baishi Mountain, and the following conclusions are drawn:

- (1) Based on the analysis of the results of multichannel transient surface wave test, superheavy (N_{120}) cone dynamic sounding test, and on-site screening test, the geological environment conditions of the uneven gravel soil on the site are further explored. The strata within the range from 0 to 15 m in Baizhi Mountain can be divided into four layers, i.e., cultivated soil, sandy soil, silty clay, and gravel soil, with depths of 1 m, 2 m, 3 m, and 9 m, respectively.
- (2) The axial force and shaft skin resistance of pile body in gravel soil are the same as those in other geological structures. At the initial loading step, the axial force and shaft skin resistance of pile body play a smaller role. With the increase of loading steps, the axial force and shaft skin resistance of pile body increase correspondingly. Under the same loading step, the axial force of the pile body and the shaft skin resistance do not mobilize in the same time and gradually transfer from the upper part to the lower part of the pile body, and the upper part of the pile body reaches the limit value before the middle and lower parts. As for the solid-bottom pile, the soil at the pile tip will strengthen the shaft skin resistance of the pile.
- (3) According to this experimental study, it is suggested that similar projects should adopt concrete manually bored cast-in-place piles with a diameter of 900 mm and a length of 15m, the wall-protecting thickness shall not be less than 175 mm, and the grade of concrete cannot be lower than C35. The pile end can be widened or not, and the pile bottom shall be free of sediment.

Data Availability

All the data in this paper are obtained from tests in this study, and no other data were used to support this study.

Conflicts of Interest

The authors declare that, with regard to the publication of this paper and the funding for publishing it, there are no conflicts of interest.

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