Research Article

Field Static Load Tests of Post-Grouted Piles under Various Failure Conditions

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1. Introduction

During the construction of a bored pile, soil is removed by using machines and hence the soil residue remains in the drilled hole. This leads to a decrease in the end resistance capacity of the pile and an increase in pile settlement. Because of these soil deposits at the base of hole, it has been found that the capacity of some piles from a mansion was much lower than the designed requirement. It has also been found that the settlement of these piles was much greater than that of the treated piles through performing the SLTs in Taiyuan City, China [1]. This particular problem can be solved by applying the slurry support technology. However, it is reported that, sometimes, this slurry support admixture layer (bentonite or polymeric material, soils, and water) decreased the friction resistance between soil layers and concrete piles, which consequently decreased the ultimate bearing capacity of the piles. It is reported that this admixture layer or composite layer decreases 30% to 40% of the pile bearing capacity [2].

The mentioned problem referring to the decreasing bearing capacity caused by the admixture layer can be solved by applying the post-grouting technology. During this grouting process, cement admixture is pressured down and the slurry support layer is then forced out [3, 4]. Grouting equipment comprises a flat jack system, which consists of grout delivery pipes connected to a steel plate with a rubber membrane, and a sleeve-port system, which consists of two to four U-tubes installed at the bottom of the pile. This U-tube is covered by rubber and can be arranged in various configurations [5].
Numerous studies have been conducted to determine the effectiveness of this technology. Field SLTs were conducted to compare the behaviours of one base grouted pile and two conventional slurry stabilized piles. These tests provided the load transfer characteristics and clarification of the mechanism of the base grouted concrete pile. They also provided the correlation among cement consumption, the number of grouting stages, and the volume of a pile shaft [6]. The diameter of piles with grouting techniques ranges from 0.4 m to approximately 2.5 m. The tests for two piles, with a pile diameter of 1.5 m, were conducted at Paksey Bridge over the Padma (Ganges) River in western Bangladesh [7]. The construction of grouted piles with a large diameter (2.4 m) commenced with The Pinnacle—a 290 m high skyscraper in London, England [8].

Besides analytical methods, the finite element method is one of the most appropriate methods to simulate the soil-structure system such as modelling the seabed structure [9]. Also, numerous modelling methods have been used to determine the bearing capacity of grouted piles. Recent test results and numerical simulation methods have shown that post-grouted concrete piles can double the ultimate capacity of a defected pile, i.e., a traditional pile’s capacity increases 20% [10]. Several projects with the application of base grouting techniques were provided by Sinnreich and Simpson [11]. In their paper, they used the O-cell test method, utilizing a bidirectional axial compressive load, to determine the shaft and end bearing capacity. However, the results were inconclusive because some projects illustrated increased grouted pile capacity and some projects did not.

Pressure-grouted piles or post-grouted piles have been successfully employed around the world for about 40 years [12]. However, previous grouting pile investigations concerned base grouting; therefore, research into shaft grouting is very limited. Moreover, research has seldom considered the ultimate uplift bearing capacity of grouted bored piles. As Sinnreich and Simpson (2013) stated, further research into the mechanics of grouting bored piles is needed, as some results of ultimate bearing capacity between grouted and nongrouted piles have been contradictory. The ultimate bearing capacity of a post-grouted pile is mostly determined by interpretation of nonplunging curve result. Other investigations paid attention to the methods of grouting to find out the best way to increase the bearing capacity under various subsurface conditions [6]. Some of the researchers take the load transfer mechanism into account. However, the SLTs of these piles under failure load are rarely conducted, and the research of these piles’ behaviours is limited.

Primarily, there are three types of pile failure conditions. The most common one is the failure of the pile head. During bored pile construction, the coarse aggregate of concrete is dense at the lower part of drilled hole and flouted slurry exists from the upper part, which is mainly the admixture of cement, fine sand, and water (ranging from 0 to 2 m at top of pile). The strength of this flouting slurry is lower than the designed strength, thus the upper part of cured concrete with low strength is always removed by a cutting machine. However, if the cutting is inefficient, the pile that contains the concrete with low strength will fail first, resulting in pile failure. The second common one is the punching failure. If the designed loading is overestimated, the end bearing stratum cannot resist the transferred loads and the pile will be pushed down immediately after the shaft resistance fully developed. Sometimes, unpredictable geotechnical conditions may also lead to the punching failure; for example, the karst cave exists around the pile. The third failure condition is the eccentricity of the pile.

In order to investigate the behaviours of post-grouted piles with these failure conditions, four post-grouted concrete piles were cast and the vertical compressive static load tests were conducted. The damage location of the concrete pile and the load transfer of the pile under punching failure are researched. This paper also provides traditional result presentation for the pile behaviour determination as well as the double tangent and Chin’s methods, which are used for ultimate capacity determination.

2. Subsurface Conditions

The construction site is in the western Jinan city, Shandong Province, China. The subsurface exploration was performed through laboratory tests and in situ tests. The in situ tests such as standard penetration tests (SPT) and laboratory tests such as consolidation tests, direct shear tests, and triaxial tests (UU) were conducted based on the local standard of Code for Investigation of Geotechnical Engineering [13] and the Chinese Code of Standard for Soil Tests Method [14], respectively. Based on the borehole logs and soil samples, 10 layers were discovered in the study area as follows:

(1) Fill, yellowish-brown, plastic clay with ash concrete and brick crumbs, and thickness ranges from 0.6 to 3.4 m. (2) Silty clay, yellow, loess-shaped, saturated, and thickness ranges from 1.2 to 3.2 m. (3) Silty clay, yellowish-brown, plastic, and thickness ranges from 0.4 to 4.4 m; (3-1) Clay, grey-brown to beige, plastic, and thickness ranges from 1.2 to 2.5 m. (4) Clayey silt, containing ferric oxide, and thickness ranges from 0.6 to 1.6 m of 5 m. (5) Clay, brown, partially containing loess-doll, particle size between 10 and 20 mm, and thickness ranges from 1.2 to 4.1 m; (5-1) Silty clay, yellowish-brown, plastic, containing iron and manganese oxides, and thickness ranges from 3.7 to 6.8 m. (6) Silty Clay, yellow to pale brown, plastic, and thickness ranges from 3.4 to 6.2 m. (7) Silty clay to clay, pale brownish red, plastic stiff, and thickness ranges from 12.3 to 34.2 m; (7-1) Clay, pale brownish red, very stiff, containing gravel with particle size between 20 and 30 mm.

The water level is discovered 2.90 to 3.75 m underground, and the excavation depth is 5–6 m. As shown in Figure 1, three piles with labels of P80, P60, and P40 are next to building 6#. These piles are of the same diameter and length; Pile 12 that is near building 3# has a larger diameter and length. Based on the borehole logs in the study area, as is shown in Figure 1, it is discovered that the subsurface conditions near building 6# are similar. The soil parameters of the test piles are summarized in Figures 2 and 3.
Figure 1: Plan view (not to scale).

Figure 2: Subsurface conditions of P80/P60/P40.
3. Pile Description

3.1. Test Pile Preparation. There are 4 piles designed to be tested. As depicted in Table 1, after curing the concrete, the pile length of P40, P60, P80, and P12 were 32 m, 30 m, 32 m, and 37 m, respectively (before pile cut). After cutting off 2 meters from the pile head as shown in Figure 4 (P40, P80, P12), all three piles were in the same dimensions of length of 30 m. Except for P12, the diameter of the other three piles was 600 mm. Because no cutting was needed for P60, the top 2 meters of the concrete did not achieve the required strength of C35. Note that, after curing the concrete, the strength of concrete from the upper pile was from C25 to C35, which was found by conducting concrete rebound tests by using a resiliometer.

3.2. Designed Ultimate Bearing Capacity. The Technical Code for Testing of Building Foundation Piles [15] has depicted some of the methods for determining the ultimate bearing capacity of pile foundation. The equations used in this project are illustrated in equations (1) and (2), which are used for the traditional pile foundation and post-grouted pile foundation, respectively. Equation (1) represents the ultimate bearing capacity of single pile without any grouting application; hence, the total capacity \( Q_{sk} \) is the sum up of shaft capacity \( Q_{sk} \) and end capacity \( Q_{pk} \). The values of side stress \( q_{sk} \) and end stress \( q_{pk} \) are relating to the types of soils, state of soils (such as void ratio, liquid index, SPT N value), types of pile foundation, and construction method. Different to equation (1), the total capacity of pile as demonstrated in equation (2) includes the extra resistances caused by shaft grouting strengthening \( Q_{sk} \) and base grouting strengthening \( Q_{pk} \). The strengthening coefficients of shaft \( \beta_{sk} \) and base \( \beta_{pk} \) are relating to the types of soils.

\[
Q_{sk} = Q_{sk} + Q_{pk} = q_{sk} l_{i} + q_{pk} A_{p},
\]

\[
Q_{sk} = Q_{sk} + Q_{pk} + Q_{gpk} = q_{sk} l_{i} + Q_{gsk} l_{g} + \beta_{pk} q_{pk} A_{p},
\]

where \( Q_{sk} \) is the ultimate bearing capacity of single pile; \( Q_{sk} \) is the shaft capacity without grouting strengthening; \( Q_{sk} \) is the base capacity without grouting strengthening; \( q_{sk} \) is the shaft resistance of each soil layer; \( q_{pk} \) is the end resistance from the pile toe; \( A_{p} \) is the end area from the pile toe; \( Q_{gsk} \) is the shaft capacity with grouting strengthening; \( Q_{gpk} \) is the base capacity with grouting strengthening; \( l_{g} \) is the shaft resistance without grouting strengthening; \( A_{p} \) is the perimeter of cross section; \( l_{i} \) is the thickness of layers within soil strengthening part; \( \beta_{sk} \) is the strengthening coefficients of shaft; and \( \beta_{pk} \) is the strengthening coefficients of base.

The capacity of single piles based on the in situ method and API method are also considered in this case study. Contrary to the JG94, the in situ method considers the SPT N value and the API method considers a coefficient, which is...
related to the overburden pressure. The ultimate bearing capacity of these four piles is summarized in Table 2. It can be seen that the total capacity obtained using the in situ method is conservative, and the capacity gained using the JGJ94 and API method are similar. It can be easily understood that the tested pile fails when maximum applied loads are greater than the calculated ultimate bearing capacity. To perform the failure tests on grouted piles, the designed maximum applied loads for SLTs are determined as 8,400 kN (>7465 kN) and 5, 200 kN (>4577 kN) for P12 and P80, P60, P40, respectively. For security consideration, the reaction weights (1.2 times the designed testing loads) are defined to be 10,080 kN (P12) and 6,240 kN (P80/P60/P40) for the testing piles. As illustrated in Figure 1, P80 (standard pile) is located underneath the building #6, this pile test is the proof test (not to be destructed). The others, however, are designed to load until failure occurs.

\[ q = \frac{q_{base}}{E} (1 - \nu^2) I_s, \]  
\[ q = \frac{T_{bi}}{\pi (D_0/2)^2}, \]
\[ G = \frac{E}{2(1 + \nu)}, \]
\[ K_{bi} = \frac{T_{bi}}{q_{base}} = 2 \frac{D_0 G}{(1 - \nu)}, \]

where \( D_0 \) is the diameter of the pile foundation, and \( E, G \), and \( \nu \) are the Young’s modulus, shear modulus, and Poisson’s ratio, respectively.

For the pile shaft, the settlement or deformation \( s(r) \) is related to the distance from the central line of the pile foundation to the point at infinity (distance \( r \) from 0 to infinity). When this distance is equal to the pile diameter \( r = D_0 \), the maximum \( s(r) \) can be obtained from the soil-pile interface; when this distance is far away from the central line \( (r = r_{max} = +\infty) \), the soil deformation is zero. In this paper, the \( r_{max} \) is assumed to be the length of the pile \( (L_p) \) because the soil settlement from this distance is very small, which can be neglected. For a pile segment with length of \( L_s \), when vertical loads are applied, the shear force from the pile shaft should be equal to the soil shear resistance: \( 2\pi (D_0/2) L_s \times \bar{\tau}_0 = (2\pi L_s) \bar{\tau}, \) which gives

\[ \bar{\tau} = \frac{(D_0/2r)}{\bar{\tau}_0}, \]

where \( \bar{\tau}_0 \) is the average shear stress acting over the element length \( L_s \), and the \( \bar{\tau} \) is the average shear stress of the soil, which results in the soil deformation.

The shear strain \( (\gamma) \) of the soil block can be represented as \( \gamma = ds/dr \). Also, it can be represented as the ratio between average shear stress \( (\bar{\tau}) \) and average shear modulus \( (G_{si}) \), as illustrated in

\[ ds = \frac{1}{r} \left( \frac{D_0 \bar{\tau}_0}{2G_{si}} \right) dr. \]

By integrating equation (8), we obtained

\[ \int_s^{0} ds = \left( \frac{D_0 \bar{\tau}_0}{2G_{si}} \right) \int_{L_s/2}^{L} \frac{1}{r} dr. \]

The soil displacement \( (s_{shaft}) \) near the pile shaft is
Table 2: Ultimate bearing capacity of piles.

| Standards         | Shaft resistance | End resistance | Total capacity (kN) | Designed test load | Reaction weight | Pile label |
|-------------------|------------------|----------------|---------------------|--------------------|----------------|------------|
| JGJ94 (nongrouting)¹ | 4,973            | 804            | 5,777               | 8,400              | 10,080         | P12        |
| In situ method    | 3,610            | 1,055          | 4,665               |                    |                |            |
| API method        | 4,285            | 904            | 5,190               |                    |                |            |
| JGJ94 (grouting)² | 5,696            | 1,768          | 7,465               |                    |                |            |
| JGJ94 (nongrouting)¹ | 3,156            | 382            | 3,538               | 5,200              | 6,240          | P80/P60/P40 |
| In situ method    | 2,669            | 466            | 3,135               |                    |                |            |
| API method        | 3,335            | 203            | 3,538               |                    |                |            |
| JGJ94 (grouting)² | 3,699            | 877            | 4,577               |                    |                |            |

Note: JGJ94 (nongrouting)¹ and JGJ94 (grouting)² represent the ultimate bearing capacity based on equations (1) and (2), respectively.

\[ s_{\text{shaft}} = \left( \frac{D_0T_0}{2G_{\text{EI}}} \right) \ln \left( \frac{2r_m}{D_0} \right). \]  

(10)

Based on the force along the pile shaft \( T_{\text{si}} = \pi D_0 L_{\text{si}} t_{\text{si}} \), the soil shaft stiffness (ratio between shaft force and shaft soil displacement) can be determined as follows:

\[ K_{\text{si}} = \frac{T_{\text{si}}}{s_{\text{shaft}}} = \frac{2\pi L_{\text{si}} G_{\text{EI}}}{\ln (2r_m/D_0)}, \]  

(11)

If the pile foundation is separated into small segments (length of \( L_{\text{si}} \), cross-sectional area of \( A_{\text{i}} \), pile’s Young’s modulus \( E_{\text{i}} \)), we can define the location where the load applied from the pile head (first segment) as node 1 with corresponding force of \( T_1 \) and the position where the transferred load from the bottom of the segments as node 2, 3, 4, . . . , \( i \) with corresponding force of \( T_2, T_3, T_4, \ldots, T_{i+1} \). The force difference between two nodes can be obtained in terms of average shortening of pile as well as shaft stiffness as

\[ T_i - T_{i+1} = K_{\text{si}} \left( \frac{\Delta z_i + \Delta z_{i+1}}{2} \right). \]  

(12)

\( (\Delta z_i - \Delta z_{i+1}) \) can be obtained based on the elastic compression of the pile element with average force in the segment:

\[ \Delta z_i - \Delta z_{i+1} = L_{\text{si}} \left( \frac{(T_i + T_{i+1})}{2E_{\text{i}}A_{\text{i}}} \right). \]  

(13)

Also, the base force can be obtained by the base settlement times the base stiffness:

\[ T_b = K_{\text{bi}} \Delta z_b. \]  

(14)

In brief, for a given applied load from the pile head \( (T_i) \) with the corresponding pile head displacement \( (\Delta z_i) \), which is obtained from the static load test as well as the known parameters of soil and pile, based on the above equations, the transferred load underneath the pile head can be obtained.

4. Field Tests

The SLTs were conducted based on the Chinese Code of Technical Code for Testing of Building Foundation Piles [15]. The set-up of the static load test is provided in Figure 5, the weighted platform was selected instead of cast reaction piles for economic consideration. The low speed maintenance methods were used in this case study. The applied load was maintained until the rate of axial movement did not exceed 0.1 mm. After each load was applied, recording the vertical movement of piles with the time intervals of 5, 10, 15 minutes, and 30 minutes was required if accumulated time exceeded 1 hour.

For piles of P40, P60, and P80, the loading started from 1,040 kN and later loading was applied with 520 kN increments. The corresponding vertical settlements under each loading were recorded by using 4 automatic dial gauges. For the piles that achieve the maximum loading of 5,200 kN, the applied loads consecutively decreased with a decrement of 1,040 kN. The jacks terminate until 0 kN, but the dial gauges continue to record the settlement until the soil-pile system being stable. For the pile P12, the loading started from 1,680 kN and later a loading of 840 kN was applied till the maximum loading reached.

5. Observation of Pile under Failure Condition

5.1. Piles with Inadequate Concrete Strength. As shown in Figure 6(a), a concrete crack in P60 occurred when the loading was at 3,640 kN. The crack was discovered at the pile head, and the maximum width of the crack was up to 30 mm. Through a low-strain integrity test, this pile was found to be broken at the depth of 1.2 m underground. After digging out the pile and breaking the concrete from pile head, it was found that the reinforcement was bent outside at a depth of 0.9 to 1.2 m, as shown in Figure 6(b). This occurred because, after the concrete was damaged, the reinforcement tried to resist the vertical loading. The reinforcement inside the pile is not bent because the concrete core resisted this deflection.

5.2. Piles under Eccentric Loads. As shown in Figure 7(b), the pile P40 was tested with an eccentricity of 200 mm. After a load of 4,680 kN was applied, the pile seemed to successfully resist the loading; however, the concrete actually crushed at the compression zone, and the reinforcement at this zone (Figure 7(a)) tried to resist this compressive load, yet failed, and the orientation was outside. At the same time, the concrete ruptured in the tensile side, but the reinforcement in this side provided tensile resistance for a couple of minutes and failed later. There was no bent reinforcement discovered in this zone (Figure 7(c)).
5.3. Failure due to Inadequate Soil Rigidity. For the pile P12 that suffered punching failure, the concrete was not damaged. After a load of 8,400 kN was applied, nothing occurred until 115 min and then the pile was suddenly driven into the ground. This phenomenon can be primarily explained by the fact that the shaft resistance fully developed, and, after 115 min, the loading transferred to the pile tip. As a result of the inadequate stiffness of the clay-bearing stratum, the pile was plunged into the ground. As shown in Figure 8, cracks up to 100 mm in the ground were discovered.
6. Test Results and Discussion

6.1. Piles with Achieved Design Requirements. The test results of the pile P80 are provided in Figures 9–11. The interpretation results based on the double tangent method, offset method, and Chin’s method are provided in Figures 12(a)–12(c), respectively. From the load-settlement curve (Q-s curve) as shown in Figure 9, the ultimate loading was conservatively determined as 4,680 kN, with a corresponding maximum settlement of 4.9 mm. For some local standards, the ultimate bearing capacity could be determined as 5,200 kN because the corresponding settlement of 7.88 mm was relatively small. According to the Chinese Standard JGJ 106-2014, Clause 4.4.2.1, through analyzing the s-lgQ results at each loading, the capacity of the pile could be determined where there was a deep downward curve occurring at the end of s-lgQ curve. As shown in Figure 10, there were two downward trends from the loading stages of 4,680 and 5,200 kN, but the settlement remained stable in the end. Hence, the capacity of the pile was determined as 5,200 kN, or 4,680 kN for the conservative consideration. Based on the s-lgQ curve, the ultimate loading was determined as 4,680 kN, with difficulty (Figure 11).

As stated in AASHTO [17] and FHWA [18], the double tangent method is more commonly used for drilled shafts. Based on plotting two tangent lines, the intersection point that presents the ultimate bearing capacity of P80 was determined to be 3,900 kN, as shown in Figure 12(a). Through the Q-s curve, elastic line and offset line were plotted. As shown in Figure 12(b), there was no intersection between the offset line and curve from the loading stages, which indicated that the ultimate bearing capacity of P80 was over the maximum applied load of 5,200 kN. When plotting the load/settlement versus settlement, the result indicated a line with a gradient of 0.0002. Based on Chin’s method, the ultimate bearing capacity of P80 was determined to be 5,000 kN, as shown in Figure 12(c).

6.2. Failure due to Inadequate Concrete Strength. The Q-s curve, s-lgQ curve, and s-lgQ curve of P60 are provided in Figures 13–15. A settlement of 12 mm was discovered for a loading of 3,640 kN, and there was an extremely increasing trend when the next loading applied; the ultimate bearing capacity was determined to be 3,640 kN for P60 based on the
Figure 11: Q-s curves of the tested piles.

Figure 12: Interpretations of (a) double tangent method; (b) Davisson’s offset method; and (c) Chin’s method.
Q-s curve (Figure 13). As shown in Figure 14, there was a dramatic decreasing trend when the loading of 4,160 kN was applied, which showed increments in settlement with increasing time, and this represented the failure criteria of the soil. So, the ultimate bearing capacity of P60 was then determined to be 3,640 kN. This figure also provides valuable information illustrating the pile failure behaviour. As shown in Figure 14, when the load of 3,640 kN was applied, the concrete broke from the upper part of the pile and the settlement increased. Later, under this vertical load, the pile foundation could still resist the applied load, and thus the settlement data remained stable in practice, when finds a curve like the 3120 kN-curve, the pile is expected be broken, checking the pile integrity is indispensable, thus low-strain detection is strongly recommended to perform. As shown in Figure 15, a point with a corresponding load of 3,640 kN was found to be the ultimate load because, after this point, the settlement of the pile head decreased dramatically.

Interpretations of the Q-s curve based on the double tangent method, Davisson’s offset method, and Chin’s method are provided in Figures 16(a)–16(c). As shown in Figure 16(a), the intersection of the two tangent lines was determined and the corresponding load was found to be 3,600 kN. For Davisson’s offset method, because this pile was not homogeneous (concrete strength from the top 2 m was inadequate), by using the concrete modulus of 31,500 N/mm² to determine the elastic line (P/E), as shown in Figure 16(b), the ultimate capacity was determined to be 3,600 kN. However, because the concrete modulus from the upper part was 30,000 N/mm², the actual capacity should be slightly over 3,600 kN. By using the data obtained during the SLT before failure occurred and then plotting the settlement/load versus settlement, a gradient of 0.0003 was discovered, as shown in Figure 16(c). Finally, the bearing capacity of P60 was determined to be 3,333 kN.

6.3. Failure due to Eccentricity. For the pile suffering eccentric loading, the typical results are presented in Figures 17–19. As shown in Figure 17, after the loading of 4,680 kN was applied, a huge increase in settlement was discovered. The ultimate capacity was then determined to be 4,160 kN. As shown in Figure 18, after loading of 4,160 kN was applied, the s-lgQ curve demonstrated that the soil settlement was relatively stable with increasing...
time; however, after the next loading was applied (4,680 kN), there was an extreme increase in settlement, which represented the failure condition. The ultimate bearing capacity was then determined as 4,160 kN. It is worth noting that there are two “drops” discovered under the load of 4,680 kN. The first settlement drop is referring to the crush of concrete from the compression zone (around 14 mm, 6 mm to 20 mm settlement change) and the second drop refers to the failure of the pile-soil system. As shown in Figure 19, after the loading of 4,160 kN was applied, a dramatic downward trend was discovered, which represented the ultimate condition. The ultimate load was then determined as 4,160 kN.

The interpretations based on the double tangent method, Davisson’s method, and Chin’s method are illustrated in Figures 20(a)–20(c), respectively. The intersection of two tangent lines was discovered from Figure 20(c), and the ultimate bearing capacity of P40 was then discovered to be 4,190 kN. As illustrated in Figure 20(b), the intersection between the offset line and Q-s curve was determined and the ultimate capacity of P40 was obtained as 4,250 kN. As shown in Figure 20(c),

Figure 16: Interpretations of (a) the double tangent method; (b) Davisson’s offset method; and (c) Chin’s method.
Figure 17: Q-s curve of P40.

Figure 18: s-lgt curve of P40.
Figure 19: s-\lg Q curve of P40.

Figure 20: Interpretations of (a) the double tangent method; (b) Davisson’s offset method; (c) Chin’s method.

\[ y = -4E^{-0.05}x + 0.0015 \]

\[ R^2 = 0.1502 \]
Figure 21: Q-s curve of P12.

Figure 22: s-lgt curve of P12.
Figure 23: $s$-lgQ curves of P12.

Figure 24: Continued.
Chin’s method was not applicable to the piles suffering the failure caused by eccentric loading.

6.4. Punching Failure. The typical presentations are provided in Figures 21–23. For the pile P12, the capacity was determined to be 7,560 kN, as shown in Figure 21, because, after this 7,560 kN point, the Q-s curve indicated a dramatic downward trend, which illustrated the failure condition. As shown in Figure 22, under the loading of 7,560 kN, the pile was relatively unstable, but the pile failed when 8,400 kN was applied. The ultimate capacity of this pile was determined to be 7,560 kN, based on the s-lgQ curve. In addition, from the s-lgQ curve, the ultimate bearing capacity was determined to be 7,560 kN (Figure 23).

The results based on the double tangent method, Davisson’s method, and Chin’s method are presented in Figures 24(a)–24(c), respectively. As shown in Figure 24(a), the intersection between two tangent lines was found, and the ultimate bearing capacity was determined to be 7,560 kN. For Davisson’s method as shown in Figure 24(b), the intersection between the offset line and Q-s curve was discovered, and the capacity was determined to be 7,900 kN. By plotting the data obtained from the tested pile before failure occurred, based on Chin’s method, a function with a slope of 0.001 was determined (Figure 24(c)) and the ultimate bearing capacity was obtained, which was equal to 1/0.0001 = 10,000 kN.

A summary of these pile capacities is provided in Table 3. It can be seen that the ultimate bearing capacity obtained from traditional methods and interpretations are generally close. For the standard pile, comparing with the ultimate bearing capacity of the grouted pile as depicted in Table 1 (4,577 kN), the tested pile’s capacity is overall 4,777 kN, which illustrates the design method provided by JGJ94 (grouting) is ideal. It should be noted that, in this paper, the diagram based on Chin’s method is determined using the test data before failure occurred (the maximum applied loads with corresponding settlement are not used in the plotted Chin’s diagram). The reason to do this is to check if Chin’s method can provide appropriate results under nonfailure condition or if this method can be used for capacity determination under proof tests (PTs). As shown in Table 3, the capacity obtained by Chin’s method is close to the results acquired from the other methods, which confirms that Chin’s method can be appropriately used for pile capacity prediction when the proof tests are performed. Also, it is found that Chin’s method cannot be used for piles with eccentricity failure.

Based on the equations (6) and (10)–(13), by including the diameter of the pile, thickness of soil layers, modulus and Poisson’s ratio of concrete and each soil layer, as well as the applied loads from the pile head with corresponding pile head displacement, the load transfer mechanism is determined as shown in Figure 25 (note that \( r_m \) is assumed to be equal to the pile length \( L_p \) to calculate the shaft stiffness). As shown in Figure 25, the transferred load decreased along the pile shaft due to the development of the soil resistance. Also, it can be found that after the maximum load of 7,560 kN was applied, the transferred load from the pile end was determined as 1,768 kN. Similar to the data in Table 2, for P12, when the total load of 7,465 kN was applied, the end bearing
was determined as 1,768 kN; if more load is applied, the bearing stratum will fail and punching failure will occur. The limitation of this research is that the cable of the installed steel sister rebar (strain gauge) was broken while cutting the pile head; thus, the acquired data are uncertain, and further research is required for load transfer mechanism investigation referring to the grouted pile’s punching failure.

7. Conclusion and Recommendation

This paper provides the pile behaviour investigation with consideration of inadequate concrete strength, punching failure, and eccentricity loading. The compressive static load tests were conducted, and the results of load-settlement, settlement-lg (load), and settlement-lg (time) curves were analyzed for capacity determination. Furthermore, the interpretations of the double tangent method and Chin’s method are provided. Thus, the following conclusions are made:

(1) For the pile with low concrete strength, which is caused by inappropriate construction, the pile’s concrete will crack at the pile head, which may extend up to 30 mm, and the concrete will crush at a certain depth. The reinforcement in the crushed concrete zone will be forced outside.

(2) For the pile suffering eccentricity failure, the concrete will crush in the compressive zone, and in this zone, the reinforcement will be forced outside. The steel in the tensile zone resists loading after the concrete ruptured, and consequently the pile fails.

(3) For the pile suffering punching failure, there is no concrete cracks discovered, but the ground soil crack can be extend up to 50 mm, the load transfer mechanism illustrates that there will be a fully development of the shaft resistance in one loading stage, and after more loading applied (next loading stage), the loads will transfer to the pile tip and consequently lead to the immediate huge settlement of pile.

(4) The traditional methods of Q-s, s-lgQ, and s-lg t and the interpretations of double tangent and Chin’s methods can be used for providing capacity results. For Chin’s method, the test data used are obtained before failure occurred, and it is concluded that Chin’s method can be used for ultimate capacity determination under nonfailure condition or under proof tests. Furthermore, Chin’s method should not be used for capacity determination for the tested pile suffering eccentric failure.

Data Availability

The excel data of field tests used to support the findings of this study are available from the first author or the corresponding author upon request.

Conflicts of Interest

The authors declare that there are no conflicts of interest regarding the publication of this paper.
References

[1] C. Shi, "The application of the pile-end mud-jacking technique used in the construction of bored caisson pile," Science and Technology Information Development & Economy, vol. 15, no. 23, pp. 293–296, 2005, in Chinese.

[2] X. Li, K. Xie, G. Zeng, and X. Hou, "Research of bored pile slurry effect created during construction," Structural Construction, vol. 30, no. 5, pp. 21–23, 2000, in Chinese.

[3] J. Zhou, X. Zhang, H. Jiang, C. Lyu, and E. Oh, "Static and dynamic load tests of shaft and base grouted concrete piles," Advances in Civil Engineering, vol. 2017, Article ID 2548020, 11 pages, 2017.

[4] J. Zhou, E. Oh, X. Zhang, H. Jiang, M. Bolton, and P. Wang, "Compressive and uplift static load tests of shaft and base grouted concrete bored piles," in Proceedings of the Twentieth International Ocean and Polar Engineering Conference, pp. 685–692, San Francisco, CA, USA, June 2017.

[5] S. Dapp, M. Muchard, and D. Brown, "Experiences with base grouted drilled shafts in the southeastern United States," in Proceedings of the Tenth International Conference on Piling and Deep Foundations, Deep Foundations Institute, Amsterdam, Netherlands, pp. 1553–1562, 2006.

[6] C. E. Ho, "Base grouted bored pile on weak granite," paper presented at the grouting and ground treatment, 2002.

[7] R. J. Castelli and E. Wilkins, "Osterberg load cell test results on base grouted bored piles in Bangladesh," in Proceedings of the GeoSupport 2004, pp. 1–16, Orlando, FL, USA, December 2004.

[8] D. Patel, S. Glover, J. Chew, and J. Austin, "The pinnacle-design and construction of large diameter deep base grouted piles in London," ground engineering, 2015.

[9] H.-Y. Zhao, J.-F. Zhu, J.-H. Zheng, and J.-S. Zhang, "Numerical modelling of the fluid-seabed-structure interactions considering the impact of principal stress axes rotations," Soil Dynamics and Earthquake Engineering, vol. 136, p. 106242, 2020.

[10] V. L. Nguyen, L. Nie, and M. Zhang, "Method cement post-grouting to increase the load capacity for bored Pile," Research Journal of Applied Sciences, Engineering and Technology, vol. 5, no. 19, pp. 4727–4732, 2013.

[11] J. Sinnreich and R. C. Simpson, "Base grouting case studies including full scale comparative load testing," in Seventh International Conference on Case History in Geotechnical Engineering, pp. 1–9, Chicago, IL, USA, May 2013.

[12] Y. P. Lai, D. T. Bergado, G. A. Lorenzo, and T. Duangchan, "Full-scale reinforced embankment on deep jet mixing improved ground," Proceedings of the Institution of Civil Engineers-Ground Improvement, vol. 10, no. 4, pp. 153–164, 2006.

[13] Ministry of Construction of the People’s Republic of China, Code for Investigation of Geotechnical Engineering (GB 50021-2001, 2001), National Standard of the People’s Republic of China, Beijing, China, 2001.

[14] Ministry of Construction of the People’s Republic of China, Chinese Code of Standard for Soil Tests Method (GB/T 50123-1999, 1999), National Standard of the People’s Republic of China, Beijing, China, 1999.

[15] Ministry of Construction of the People’s Republic of China, Technical Code for Testing of Building Foundation Piles (JGJ 106-2014, 2014), National Standard of the People’s Republic of China, Beijing, China, 2014.

[16] J. Knappett and R. F. Craig, Craig’s Soil Mechanics, Spon Press, New York, NY, USA, 8th edition, 2012.

[17] American Association of State Highway and Transportation Officials, Standard Specifications for Highway Bridges, Washington, D. C., USA, 2002.

[18] Federal Highway Administration, Z. G. Kyfor, Static Testing of Deep Foundations (FHWA SA-91-042), U. S. Department of Transportation, Washington, D. C., USA, 1991.