

Article

Bearing Characteristics of Large-Diameter Pile Foundations Based on Loading Reaction Tests Using the Tension–Compression Anchor Method

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Abstract

To clarify the bearing characteristics and load transfer mechanisms of long large-diameter pile foundations in dense silty fine sand strata within the middle and lower reaches of the Yellow River, a graded tension–compression anchor reaction loading test method was devised and implemented using a field-configured apparatus. This approach enables graded static load testing on large-tonnage long bored cast-in-place piles. Then, the relative displacement and settlement between pile and soil under vertical cyclic loading were analyzed. Finally, numerical simulations were adopted to study the settlement behavior of pile tops and ends under cyclic loads representative of beam yard operational conditions (20 cycles). Results indicate that, under vertical loading, the shaft friction resistance and tip resistance of large-diameter long bored cast-in-place piles are not mobilized simultaneously, but sequentially. The degree of shaft friction is related to the magnitude of pile top loading, soil properties, burial depth, and construction methods. The soil between piles generates vertical resistance to horizontal force-transfer rods, becoming part of the pile foundation's bearing capacity and sharing the load. Moreover, in dense silty fine sand strata, long large-diameter pile foundations exhibit pure friction pile behavior. When calculating the bearing capacity of such piles, parameters from geotechnical reports based on code-specified values should be multiplied with corresponding correction coefficients. In addition, the shaft friction resistance in dense silty fine sand layers remains under-mobilized. Limited loading–unloading cycles in permanent–temporary integrated beam yard operations do not induce significant deformation in pile foundations, indicating minimal impact on their bearing performance.

Keywords: tension–compression anchor method; combined permanent and temporary girder yard; drilled cast-in-place pile; static load test; settlement law



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1. Introduction

The rapid development of super-tall structures and long-span bridges has driven extensive utilization of large-diameter extra-long bored piles, creating new geotechnical challenges during the construction phase. Frequent material handling and temporary storage operations near completed pile foundations inevitably subject these foundations to repeated cyclic loading [1–3]. This dynamic interaction induces cumulative settlement and alters bearing characteristics through soil–pile interface degradation, potentially compromising their stability. Understanding the deformation mechanisms and load transfer behavior of piles under cyclic loading conditions, therefore, becomes critical for ensuring engineering safety and optimizing design methodologies.

Existing research on pile foundation performance has evolved through four approaches: field testing, theoretical modeling, experimental simulation, and numerical analysis. Field investigations, while providing direct empirical data, face practical limitations for cyclic loading applications owing to high costs and operational constraints. Notable contributions include full-scale static load tests on skyscraper foundations [4], post-grouting optimization studies for expressway projects [5], and large-tonnage vertical loading tests revealing shaft friction mechanisms [6]. Recent field investigations on extra-large-diameter bored piles in lacustrine deposits [7] and rooted bored piles [8] have further elucidated shaft friction mobilization patterns, while field-calibrated 3D FEM analyses [9,10] have enhanced load transfer predictions for super-long piles. Cyclic loading research has progressed through phased load (static–cyclic–static sequences) on instrumented piles [11], with international researchers validating field measurement techniques for cyclic pile loading [12]. Recent advances combine in situ monitoring with soil structure interaction analysis, demonstrating cyclic shear-induced particle rearrangement at pile interfaces [13] and quantifying deformation dependencies on loading parameters [14].

Complementing field studies, scaled model testing has emerged as a flexible alternative despite requiring strict similitude control. Systematic experiments have established correlations between cyclic load ratios and settlement accumulation [15], while laboratory simulations have identified critical factors including loading frequency, amplitude, and soil density [16]. Theoretical developments have produced simplified design formulas for cyclically loaded piles [17], though practical applications remain constrained by oversimplified assumptions. Numerical modeling has gained prominence through advanced finite element analyses incorporating parameter inversion techniques, interface sensitivity studies [18], and hybrid experimental–numerical approaches [19]. Concurrently, advanced interface constitutive models incorporating hardening–softening behavior [20] have refined pile–soil interaction simulations, and experimental investigations on axial cyclic response [21] along with semi-analytical interface-based solutions [22] have deepened the understanding of cyclic degradation mechanisms. These computational methods enable detailed simulation of load transfer mechanisms while overcoming field testing limitations.

This study addresses the specific challenge of construction-induced cyclic loading from prefabrication operations in a permanent–temporary integrated beam yard, focusing on 80 m long bridge piles in Henan Province’s Yellow River crossing project in China. Combining field load testing with calibrated finite element modeling, this research establishes a dual verification framework to: (1) characterize actual load-bearing states through instrumented pile monitoring, (2) develop numerical models via parameter sensitivity analysis, and (3) systematically investigate settlement patterns under vertical cyclic loading sequences. The integrated approach aims to bridge existing gaps between empirical observations and computational predictions for extra-long pile foundations subjected to construction-phase dynamic loads.

2. Engineering Background

2.1. Project Overview and Site Conditions

The Henan Provincial Highway Yellow River Bridge and its connecting lines constitute a critical river crossing project spanning the Yellow River in China, with a total route length of 14.07 km. They provide the most direct traffic corridor from a certain prefecture-level city in Henan Province to Shandong Province.

The project construction area is surrounded by dense farmland, resulting in limited temporary construction land. Moreover, the project site is located within the Yellow River floodplain, where the foundation bearing capacity is poor, and the individual dimensions and weight of the corrugated steel web PC composite girders are considerable. Extensive

ground improvement for the prefabrication and girder storage areas would inevitably increase construction costs significantly. Meanwhile, moving these areas to a distant location would also raise transportation costs. In response to the difficulties of locating the prefabrication yard for corrugated steel web PC composite girders, an innovative plan was proposed to adopt a “permanent–temporary integration” strategy. In other words, the temporary girder fabrication and storage areas are built on top of the constructed foundation piles. Once the girder casting and erection in the corresponding bridge segment are completed, the girder yard is dismantled, and the construction proceeds with the pier columns and superstructure above the pile caps. The repeated and cyclic loading during girder casting and storage in the girder yard can easily induce cumulative settlement and changes in bearing characteristics in the adjacent piles.

The project extends from south to north across two geomorphic units—the Yellow River floodplain and the overflow flatland—both of which are relatively flat. The main bridge and approach spans cross the Yellow River channel and its embankments on both sides. The lithology of the overflow flatland is predominantly Quaternary silty clay, silt, and sandy soils. The Yellow River floodplain includes the river’s overbank area and main channel, featuring a relatively thick layer of weak soils in the upper strata—mainly medium-dense silt, fine sand, and from fluid-plastic to soft-plastic silty clay—underlain by silt, silty clay, and sandy soils. Groundwater in the project area primarily exists as pore water in loose rock formations, classified as unconfined water. The silt and sand layers serve as the principal aquifers, recharged by atmospheric precipitation, with irrigation also affecting the water table to some extent.

To thoroughly investigate the bearing performance of large-diameter bored piles used in the Yellow River Highway Bridge and its connecting lines, single-pile vertical static load tests were conducted on site. Based on the local geological conditions, three test piles (FJS1, FJS2, and FJS3) were arranged within the project alignment, with a distance of 1600 m between Test Pile 1 and Test Pile 2, and 1250 m between Test Pile 2 and Test Pile 3. The distribution of the soil layers corresponding to the three test piles is shown in Figure 1. The soil layer parameters at the FJS1 test pile site are shown in Table 1.

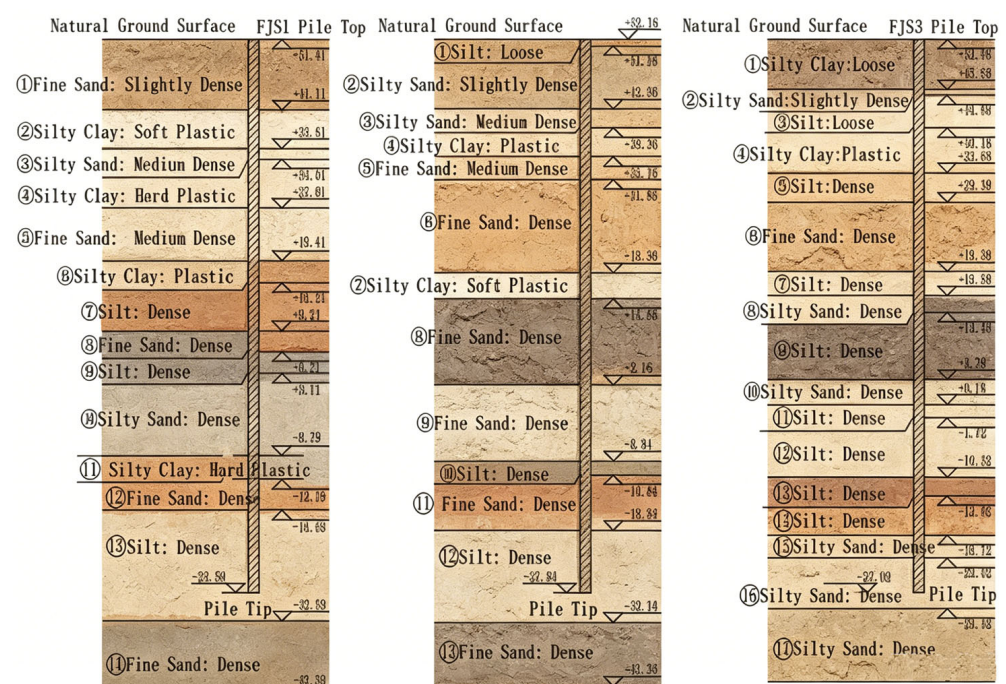


Figure 1. Soil layer distribution of test piles.

Table 1. Soil layers and physico-mechanical parameters.

Layer Number and Name	Layer Thickness (m)	State	Unit Weight/ (kN·m ³)	Compression Modulus E_{s1-2} (MPa)	Poisson's Ratio
①Fine sand	10.3	Slightly Dense	19.5	6.2	0.3
②Silty clay	5.5	Plastic	19.6	5.1	0.3
③Silt	1.6	Medium Dense	19.8	9.3	0.3
④Silty clay	7.0	Hard Plastic	20.0	7.2	0.25
⑤Fine sand	7.6	Medium Dense	19.8	7.3	0.26
⑥Silty clay	3.2	Plastic	19.8	5.2	0.3
⑦Silt	7.0	Dense	19.5	8.8	0.3
⑧Fine sand	3.0	Dense	19.8	9.4	0.25
⑨Silty	3.1	Dense	19.8	9.3	0.3
⑩Fine sand	11.9	Dense	19.9	9.5	0.25
⑪Silty clay	3.4	Hard Plastic	20.1	9.8	0.25
⑫Fine sand	4.5	Dense	20.0	10.2	0.25
⑬Silt	15.9	Dense	19.9	10.3	0.3
⑭Fine sand	11.0	Dense	20.2	10.5	0.25

Layer Number and Name	Cohesion (kPa)	Internal Friction Angle/°	Standard Side Friction Resistance q_{ik} (kPa)	Basic Allowable Bearing Capacity $[f_{ao}]$ (kPa)
①Fine sand	4	35	30	90
②Silty clay	26	14	40	120
③Silt	5	25	40	140
④Silty clay	25.5	18	50	160
⑤Fine sand	3.5	35.5	50	180
⑥Silty clay	28	17	60	200
⑦Silt	12	24	60	200
⑧Fine sand	4.8	35.8	60	230
⑨Silty	11	25	60	200
⑩Fine sand	5	36.5	65	240
⑪Silty clay	28.2	18.3	75	250
⑫Fine sand	5.2	36.8	70	250
⑬Silt	12.5	26.5	80	240
⑭Fine sand	5.5	37.5	75	260

2.2. Statistical Comparison of Test Pile Sites and Construction Quality

The stratigraphic conditions at the three test locations exhibit comparable layer sequences with moderate variability in key engineering parameters (coefficient of variation $CV < 15\%$ for layer thickness and stiffness of the major bearing strata as detailed in the statistical comparison presented below), and the pile tips are all seated in dense silt and fine sand layers.

As summarized in Table 2, the key soil layer parameters across the three test pile locations exhibit coefficients of variation (CV) generally below 15%, with the bearing layer properties (fine sand layers ⑤ and ⑧) showing CV values as low as 2.0–2.1% for the internal friction angle. The moderate variability in layer thicknesses (CV = 6.8–12.7%) is attributable to the natural spatial heterogeneity of alluvial deposits in the Yellow River floodplain. These statistical results quantitatively confirm that the stratigraphic conditions at the three test locations are sufficiently comparable to support the use of the FJS1 profile as a representative case for numerical modeling. The adoption of a single representative profile is further justified by the regional sedimentological characteristics of the Yellow River alluvial plain: the floodplain deposits are dominated by laterally extensive regionally continuous silt and fine sand layers formed by successive overbank and channel-fill sedimentation events over geologic time scales. The low CV values observed for the key bearing layers (CV = 2.0–2.1% for ϕ of layers ⑤ and ⑧) are consistent with this depositional model, and the borehole data across the 14.07 km alignment confirm the persistence of the principal stratigraphic units. The FJS1-based model thus provides a representative, rather than site-specific, characterization of the subsurface conditions along the alignment.

Table 2. Statistical comparison of key geotechnical parameters among the three test pile locations.

Soil Layer	Parameter	FJS1	FJS2	FJS3	Mean $\pm$ SD	CV (%)
Fine sand (①)	Thickness (m)	10.3	9.8	11.2	10.4 $\pm$ 0.71	6.8
Fine sand (①)	N-value (blows/30 cm)	12	10	13	11.7 $\pm$ 1.53	13.1
Silty clay (②)	Thickness (m)	5.5	6.2	4.8	5.5 $\pm$ 0.70	12.7
Silty clay (②)	cu (kPa)	26	24	28	26.0 $\pm$ 2.00	7.7
Fine sand (⑤)	Thickness (m)	7.6	8.1	7.0	7.6 $\pm$ 0.55	7.2
Fine sand (⑤)	ϕ (°)	35.5	34.8	36.2	35.5 $\pm$ 0.70	2.0
Silt (⑦)	Thickness (m)	7.0	6.5	7.8	7.1 $\pm$ 0.66	9.2
Fine sand (⑧)	ϕ (°)	35.8	35.0	36.5	35.8 $\pm$ 0.75	2.1
Fine sand (⑧)	Es1-2 (MPa)	9.4	8.9	10.1	9.5 $\pm$ 0.60	6.3

Table 3 reveals notable differences in construction quality among the three test piles. The bottom sediment thickness for FJS2 (15 mm) is approximately half that measured for FJS1 (28 mm) and FJS3 (32 mm), which directly contributes to the higher pile end resistance observed at FJS2 (4845 kN vs. 2128 kN for FJS1). The concrete filling coefficient for FJS2 (1.08) also exceeds those of the other two piles (1.02–1.03), indicating superior borehole quality and concrete placement. These construction-related factors, rather than inherent soil variability, appear to be the primary drivers of the inter-pile differences in load settlement behavior discussed in Section 4.1.

Table 3. Comparison of construction quality parameters and measured responses of the three test piles.

Construction Parameter	FJS1	FJS2	FJS3	Specification Limit
Pile diameter (m)	1.5	1.5	1.5	—
Pile length (m)	80	80	80	—
Drilling method	Rotary, slurry	Rotary, slurry	Rotary, slurry	—
Mud density (g/cm ³)	1.18	1.12	1.20	1.05–1.25 (JGJ 94)
Bottom sediment thickness (mm)	28	15	32	$\leq$ 50 (JGJ 94)
Concrete filling coefficient	1.03	1.08	1.02	$\geq$ 1.0 (JGJ 94)
Drilling duration (h)	18.5	22.0	17.0	—
Concrete pouring duration (h)	4.2	3.8	4.5	$\leq$ 6 (first batch)
Ultrasonic integrity test	Class I	Class I	Class I	Class I or II
Pile top settlement at max load (mm)	46.35	39.13	45.93	$\leq$ 60 (design)
End resistance at max load (kN)	2128	4845	3261	—
Rebound rate (%)	37.7	40.9	36.7	—

3. On-Site Tension–Compression Anchor Reaction Test

Based on the geotechnical investigation data along the route and the results of similar single-pile vertical static load tests in the vicinity, the maximum loading value for this test was preliminarily set to 4.00×10^4 kN, such that the test pile would be loaded to its ultimate bearing capacity. This static load test employed a field-configured tension–compression anchor reaction system, arranged in accordance with the relevant static load testing principles for foundation piles [23], to conduct staged loading on the large-diameter pile foundations. The apparatus is capable of uniformly providing high-tonnage reaction forces.

The steel strands, six anchor piles, and the load transfer beam (or load transfer rod) were arranged radially around a central test pile. Specifically, six engineering piles beneath a designated pier were used as anchor piles, and the middle engineering pile served as the test pile. The six anchor piles jointly provided the counteracting tension force to load the central test pile for the bearing capacity test. The test pile was embedded in a semi-infinite soil mass, with the anchor piles in contact with the surrounding soil. The top of the test pile was connected to the bottom of the load transfer beam (rod), which was, in turn, connected to the upper load-bearing system by steel strands, forming a hyperstatic structural system that worked in unison.

It is noteworthy that the horizontal load transfer beams (or rods) connecting the central test pile to the peripheral anchor piles are embedded within the near-surface soil layer. As the test pile undergoes downward displacement under vertical loading, the horizontal load transfer elements bear against the overlying soil, generating a passive vertical resistance. This soil reaction force, transmitted through the load transfer elements to the anchor piles, effectively contributes to the overall load-bearing capacity of the pile foundation system. The mobilized soil resistance depends on the embedment depth, soil properties, and the relative stiffness of the load transfer elements, and represents an additional load-sharing mechanism beyond the conventional shaft friction and end resistance.

The test pile had a diameter of 1.5 m and a length of 80 m, while each anchor pile had a diameter of 2.0 m and a length of 85 m. The anchor piles and loading apparatus together could provide a reaction force of no less than 1.2 times the estimated maximum test load, i.e., 4.80×10^4 kN. The on-site static load testing setup is shown in Figure 2. Taking FJS2 as an example, the maximum load considered was 4×10^4 kN, applied in 10 increments of 4×10^3 kN in each step.

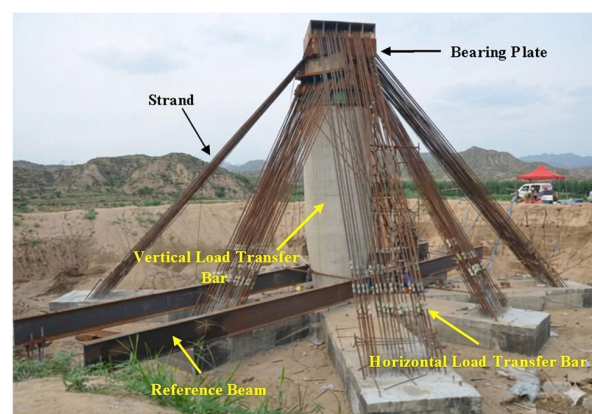


Figure 2. Tension and compression anchor method loading reaction force test.

The loading was applied step by step using hydraulic jacks with a slow maintenance method, increasing the load up to the maximum value. Subsequently, graded unloading was performed in increments equal to twice the step load. To ensure that the force transfer

during loading and unloading was uniform, continuous, and shock-free, the load variation at each stage was maintained within $\pm 10\%$ of the step load.

The settlement at the top of the test pile and the uplift of the anchor piles were measured using electronic displacement sensors, which were fixed to the reference beam via magnetic dial gauges. Meanwhile, rebar strain gauges installed on the pile reinforcement were used to measure the axial force in the pile under different loading levels, from which the pile shaft friction was calculated. Figure 3 provides a detailed illustration of this setup.



Figure 3. Settlement and axial force measurement device.

To ensure the reliability of the test results, an integrity test was conducted on the pile shaft prior to loading at the pile top. The ultrasonic transmission test results indicated that all sonic logging lines exhibited normal acoustic parameters and received waveforms, classifying the piles as Category I with sound pile shaft integrity.

4. On-Site Test Results and Analysis

4.1. Load (Q)–Settlement (S) Curve Analysis

Figure 4 shows the Q – S curves at the top of the three test piles in the static load tests, and the comparative results for in situ test pile top settlements given in Table 4. The following observations can be made: FJS1 had a maximum applied load of 3.40×10^4 kN, with a Q – S curve exhibiting a steep drop. The single-pile vertical ultimate bearing capacity (standard value) is taken as the starting point of the steep descent segment, i.e., 3.20×10^4 kN. Based on the maximum stable settlement of 46.35 mm at the top of the pile under the maximum load and a 17.46 mm rebound after unloading, the rebound rate is 37.7%. FJS2 had a maximum applied load of 4.00×10^4 kN, and its Q – S curve likewise showed a steep drop. The ultimate bearing capacity (standard value) is taken as the starting point of the steep descent segment, i.e., 3.60×10^4 kN. With a maximum stable settlement of 39.13 mm under the maximum load and a 16.00 mm rebound after unloading, the calculated rebound rate is 40.9%. FJS3 had a maximum applied load of 3.60×10^4 kN, and its Q – S curve presented a gradually varying form. The ultimate bearing capacity (standard value) is taken as the load corresponding to 40 mm of top settlement, which is 3.38×10^4 kN. Given the maximum stable settlement of 45.93 mm at the peak load and a 16.85 mm rebound after unloading, the rebound rate is 36.7%.

Table 4. Comparative analysis of in situ test pile top settlement.

Test Pile Number	Maximum Settlement (mm)	After Loaded (mm)	Maximum Rebound (mm)	Rebound Rate (%)
FJS1	46.35	28.89	17.46	37.7
FJS2	39.13	23.13	16.00	40.9
FJS3	45.93	29.08	16.85	36.7

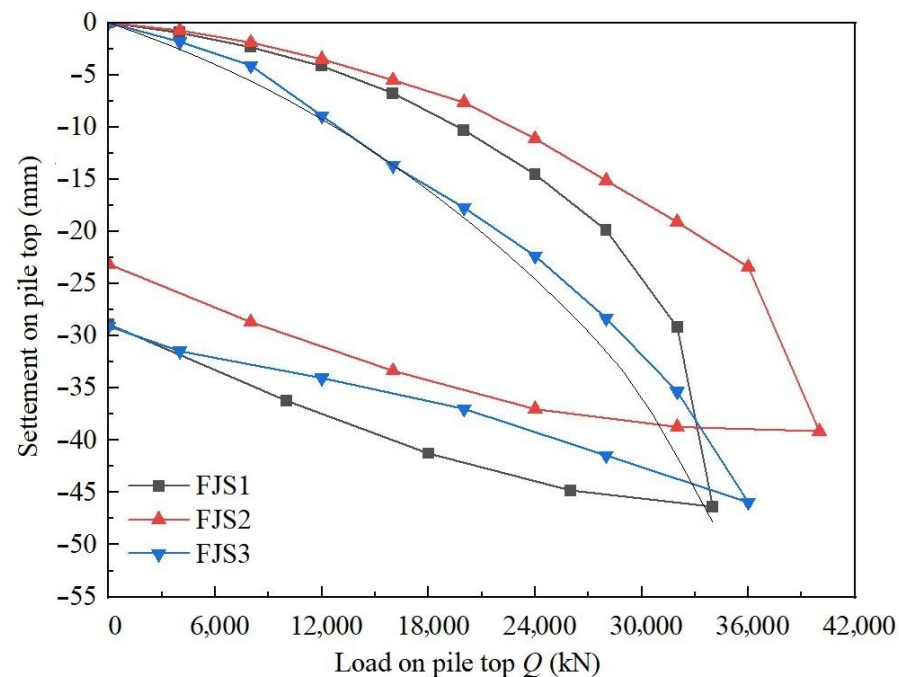


Figure 4. Q-S curve of pile top.

4.2. Analysis of Axial Force in the Pile Shaft

The pile shaft is assumed to have a constant cross-sectional area along its length and to behave as a linearly elastic material. It is also assumed that the concrete and the rebar strain gauges on the same cross-section satisfy strain compatibility. That is, under the same load level, the concrete strain in the test pile, the longitudinal reinforcement strain, and the strain indicated by the strain gauges are assumed to be equal. Therefore, the axial force in the pile shaft can be calculated using the simplified deformation Equations (1) and (2), derived based on elastic theory:

$$P_{zi} = A_c E_c \varepsilon_c + A_s E_s \varepsilon_s = \frac{A_c E_c + A_s E_s}{A_g E_g} P_{gi}, \quad (1)$$

$$P_{gi} = K(f_i^2 - f_0^2), \quad (2)$$

where P_{zi} is the axial force at a particular cross-section of the pile shaft under the i -th load level. A_c , A_s , and A_g are the cross-sectional areas of the concrete, longitudinal reinforcement, and strain gauge in the pile shaft at the given cross-section, respectively. E_c , E_s and E_g are the elastic moduli of the concrete, longitudinal reinforcement, and strain gauge, respectively. P_{gi} is the pressure exerted on the strain gauge under the i -th load level. K is the calibration coefficient of the strain gauge. For frequency measurements, f_0 is the value measured before applying any vertical load after embedding the strain gauge, and f_i is the value at the pile top as the load gradually increases to the i -th load level.

Figure 5 shows the distribution of axial force under different load levels for the three test piles, from which the following observations can be made: the load at the top of the pile is supported by both the side friction and the end resistance. When the pile top vertical load is relatively small, the load is mainly supported by the side friction, and the axial force at the pile end is essentially zero within a certain range. As the load increases, the relative displacement between the pile and soil increases, leading to a greater contribution of side friction, while the surrounding soil at the pile end is compressed, which, in turn, increases the pile end resistance. In the early stages of loading, due to the smaller load at the pile top, the axial force has not yet been transmitted to the pile end, and the side friction is not

fully mobilized. This results in a relatively steep slope in the pile axial force curve. As the hydraulic jack continues to increase the pressure and the pile top load gradually increases, the side friction becomes more significant, causing a steady increase in the axial force along the pile shaft, with the slope of the curve becoming progressively gentler.

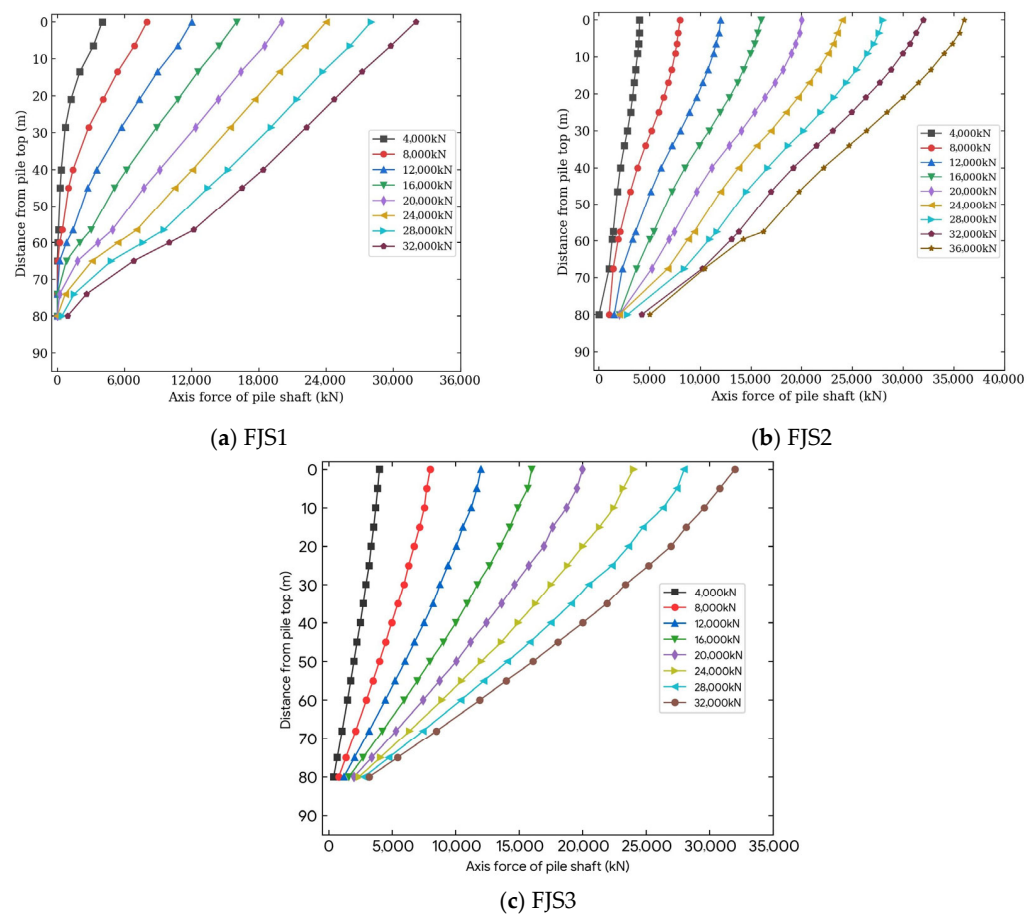


Figure 5. Axial force of test pile at various loading levels.

For the FJS1 test pile, when the pile top load is 1.20×10^4 kN or below, the pile end resistance is zero. When the load increases to 1.60×10^4 kN, the pile end resistance increases to 140.60 kN. For FJS2, when the pile top load is 4.00×10^3 kN, the pile end resistance is zero, and when the load increases to 8.00×10^3 kN, the pile end resistance increases to 142.5 kN. For FJS3, when the pile top load is 8.00×10^3 kN or below, the pile end resistance is zero, and when the load increases to 1.20×10^4 kN, the pile end resistance increases to 562.3 kN. This phenomenon is likely attributed to differences in the strata at the three test locations, as well as factors such as the construction methods (e.g., pile base sediment conditions). It also indicates that the side friction and pile end resistance of large-diameter extra-long piles do not develop simultaneously, but instead follow a certain sequence. When the maximum load for the FJS1 test pile reaches 3.40×10^4 kN, the pile end resistance is 2128 kN, accounting for about 6.3% of the total load. For FJS2, when the maximum load reaches 4.00×10^4 kN, the pile end resistance is 4845 kN, accounting for about 12.1%. For FJS3, when the maximum load reaches 3.60×10^4 kN, the pile end resistance is 3261 kN, accounting for about 9.1%. In all three cases, the pile end resistance did not fully develop, indicating the characteristics of a friction pile.

4.3. Side Friction Analysis of the Pile Shaft

The side friction q_{si} at each soil layer along the pile shaft can be calculated using Equation (3):

$$q_{si} = \frac{P_{zi} - P_{zi+1}}{\pi \cdot D \cdot l_z}, \quad (3)$$

where q_{si} is the side friction between the z -th and $(z + 1)$ -th cross-sections of the pile shaft under the i -th load level. P_{zi} and P_{zi+1} are the axial forces at the z -th and $z + 1$ -th cross-sections of the pile shaft under the i -th load level, respectively. D is the diameter of the pile shaft. l_z is the length of the pile between the z -th and $z + 1$ -th cross-sections. Using this formula, the distribution of side friction along the pile shaft for the three test piles is shown in Figure 6.

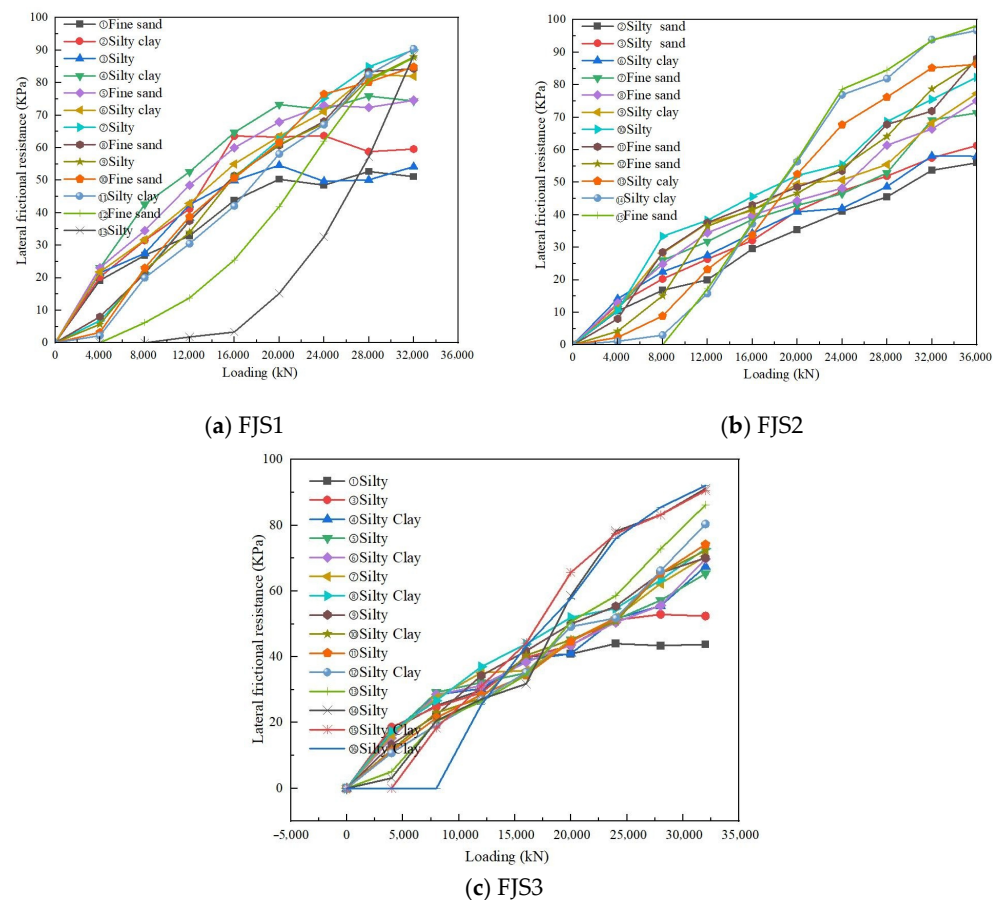


Figure 6. Variation of lateral friction resistance with applied load level for each soil layer.

Figure 6a shows that, when the pile top load reaches 2.00×10^4 kN, the side friction of the FJS1 test pile in the upper soil layers (①–④) had reached its limit and stabilized, exhibiting minor softening. In the lower layers, the side friction in the fine sand layer (⑤) and silty clay layer (⑥) reached their limits and stabilized at load levels of 2.40×10^4 kN and 2.80×10^4 kN, respectively. However, in the dense fine sand and hard-plastic silty clay layers within the lower 65 m of the pile, the side friction continues to increase as the load increases. Even after reaching the ultimate load, there is still an increasing trend. Figure 6b shows that, after reaching the ultimate load, the side friction of the FJS2 test pile in all soil layers has not yet stabilized, showing a continued increase. Figure 6c shows that, only in the upper 8 m range, the side friction of the FJS3 test pile in the fine sand (①) and silt (③) layers reaches a constant value and stabilizes when the pile top load reaches 2.40×10^4 kN. In the lower dense fine sand layer, the side friction continues to increase

with the increasing pile top load with no signs of stabilization. From the above analysis, it can be concluded that the development of side friction in the pile is influenced by several factors, including the applied pile top load, soil layer properties, burial depth, and the construction techniques of the pile foundation.

To interpret the sequential shaft friction mobilization observed in the field tests, the pile–soil interface shear mechanism is illustrated in Figure 7. This established mechanism (e.g., Bolton, 1986 [24]; Pham et al., 2024 [20]) provides a theoretical framework for understanding the observed behavior, although direct site-specific interface shear tests were not performed in the present study. In dense silty fine sand strata, the interface undergoes progressive shear evolution comprising three phases: (1) Elastic phase: interface shear stress increases approximately linearly with relative pile–soil displacement, governed by the interface shear stiffness. (2) Elastoplastic transition phase: sand particles near the interface begin to rearrange and reorient, accompanied by dilation of the dense sand fabric. The dilatancy-induced increase in effective normal stress at the interface is consistent with the stress dilatancy theory (Bolton, 1986 [24]). This dilatancy-enhanced shaft resistance mechanism aligns with recent analytical models for bored piles in sand [25] and softening-based interface formulations [20] that capture the shear stress–displacement behavior at the pile–soil contact, providing an additional shear resistance component beyond the constant-volume friction angle. (3) Critical state phase: interface shear stress reaches a plateau governed by the residual friction angle of the sand–concrete interface, with a fully developed localized shear band characterized by altered grain size distribution and fabric anisotropy. This mechanism is particularly significant in the dense fine sand layers (layers ⑤ and ⑧), where high relative density promotes pronounced dilatancy during shear, thereby enhancing the mobilized shaft friction beyond values predicted by conventional interface models. Figure 7 is a conceptual illustration and is not calibrated from site-specific interface shear test data.

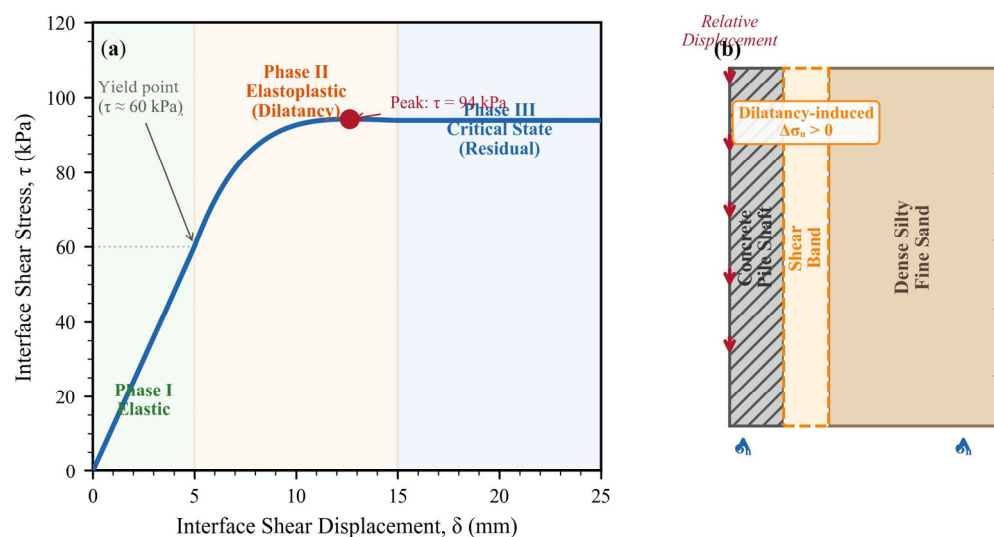


Figure 7. Pile–soil interface shear mechanism in dense silty fine sand: (a) idealized shear stress–displacement (τ – δ) curve illustrating the three-phase evolution (elastic $\rightarrow$ elastoplastic with dilatancy $\rightarrow$ critical state), and (b) schematic representation of the interface shear band showing grain crushing, reorientation, and dilatancy-induced normal stress increase.

4.4. Comparison of Pile Shaft Side Friction Test

The comparison between the side friction values corresponding to the ultimate load for the three test piles and the recommended values based on geotechnical investigation reports is shown in Table 5. From Table 5, it can be seen that the measured side friction values for

both the upper and lower soil layers of the pile shaft are greater than the recommended values based on the standards and empirical guidelines in the geotechnical report, with the maximum difference reaching up to 50%. For example, at position ⑦ in the dense silt layer for the FJS1 test pile, combined with the fact, shown in Figure 6, that the side friction in most segments has not fully developed, the difference between the measured and recommended values appears to be increasing. This indicates that the dense fine sand layers in the middle and lower Yellow River basin provide strong lateral confinement for the pile shaft. Several physical mechanisms contribute to the elevated shaft friction values. First, the angular grain morphology of Yellow River alluvial sands increases inter-particle interlocking and interface roughness. Second, the stress redistribution and lateral stress recovery during bored pile construction generates residual lateral stresses that enhance the effective normal stress at the pile–soil interface. Third, the dilatancy of dense fine sand during shear produces an additional normal stress increment. Fourth, the construction method (rotary drilling with slurry wall protection) results in a slightly rougher pile shaft surface, improving mechanical interlock. Based on these findings, for similar projects involving extra-long bored piles, it is recommended to appropriately adjust the standard values by considering the geotechnical conditions and applying a more reasonable amplification factor to ensure both safety and cost-efficiency.

Table 5. Comparative analysis of measured and recommended pile shaft side friction.

Pile Shaft Resistance of FJS1			Pile Shaft Resistance of FJS2			Pile Shaft Resistance of FJS3		
Soil Lay	Measured Geostress (kPa)	Recommended Geostress (kPa)	Soil Lay	Measured Geostress (kPa)	Recommended Geostress (kPa)	Soil Lay	Measured Geostress (kPa)	Recommended Geostress (kPa)
①Fine sand, Slightly Dense	51.1	30	②Silty clay, Slightly Dense	56.0	35	①Fine sand, Loose	43.8	30
②Silty clay, Plastic	59.6	40	③Silt, Medium Dense	61.3	40	③Silt, Loose	52.4	30
③Silty, Medium Dense	54.2	40	⑥Silty clay, hard Plastic	58.0	50	④Silty clay, Plastic	67.5	55
④Silty clay, hard Plastic	74.4	50	⑦Fine sand, Medium Dense	71.3	55	⑤Silty, Dense	65.3	55
⑤Fine sand, Medium Dense	74.6	50	⑧Fine sand, Dense	75.1	60	⑥Silty clay, Dense	69.8	60
⑥Silty clay, Plastic	82.0	60	⑨Silty, hard Plastic	77.3	60	⑦Silty sand, Dense	70.6	55
⑦Silty, Dense	90.1	60	⑩Silty clay, hard Plastic	82.3	60	⑧Silty sand, Dense	72.9	55
⑧Fine sand, Dense	84.3	65	⑪Silty, Dense	88.1	65	⑨Silty, Dense	70.1	60
⑨Silty, Dense	87.9	60	⑫Fine sand, Dense	87.2	65	⑩Silty sand, Dense	72.4	65
⑩Fine sand, Dense	84.9	65	⑬Fine sand, Dense	86.3	70	⑪Silty, Dense	74.2	65
⑪Silty clay, hard Plastic	90.4	75	⑭Silty clay, Dense	86.3	70	⑫Silty sand, Dense	80.4	70
⑫Fine sand, Dense	87.7	70	⑮Silty clay, hard Plastic	96.7	70	⑬Silty, Dense	86.2	65
⑬Silty, Dense	88.4	80	⑯Fine sand, Dense	98.1	80	⑭Silty, Dense	91.2	75
—	—	—	—	—	—	⑮Silty sand, Dense	90.6	70
—	—	—	—	—	—	⑯Silty sand, Dense	92.1	70

The recommended correction factors λ presented in Table 6 provide quantitative design guidance derived from the three test piles. For dense silt—the dominant tip-bearing stratum in this study—the recommended $\lambda = 1.35$ corresponds to a 35% increase over the code-specified standard value, which is consistent with the enhanced lateral confinement and dilatancy effects discussed above. Similar observations of shaft-dominated load transfer and limited toe-bearing mobilization have been reported for solidified soil piles in layered soft ground [26]. It is emphasized that these λ values are site-specific and should be validated through additional field load tests before application to other geological settings.

in the Yellow River basin. It is important to note that these correction factors are derived from only three instrumented test piles located within a single construction project in the middle-lower Yellow River basin. Consequently, their direct applicability to other geological environments—such as soft clay deposits, weathered rock formations, or regions with different seismotectonic settings—remains unverified. Furthermore, the arithmetic mean approach used to derive λ values reflects the central tendency of a limited dataset ($n = 3$) and does not incorporate formal statistical confidence intervals. Engineers should therefore treat these factors as preliminary recommendations requiring project-specific field validation through instrumented static load tests before adoption in design. Broader application of these correction factors across the Yellow River alluvial plain would require a regional calibration program involving additional test piles at multiple bridge sites to establish statistically reliable λ values with quantified uncertainty bounds.

Table 6. Recommended correction factors λ for side friction resistance in dense silty fine sand strata.

Soil Layer Type	Soil State	Meas./Rec. Ratio Range	Recommended λ	Applicable Condition
Silt	Medium dense	1.15–1.38	1.25	Middle-lower Yellow River alluvial silt
Fine sand	Medium dense	1.35–1.62	1.50	Floodplain fine sand, $D_r = 0.4$ – 0.6
Silt	Dense	1.22–1.45	1.35	Deep dense silt, depth > 30 m
Fine sand	Dense	1.42–1.68	1.55	Deep dense fine sand, depth > 50 m
Silty clay	Hard plastic	1.30–1.52	1.40	Over-consolidated silty clay
Silty clay	Plastic	1.18–1.35	1.25	Normally consolidated silty clay

Based on the comparative analysis in Table 6, the following tentative correction factors are recommended for side friction resistance in similar geological conditions within the middle-lower Yellow River basin: $\lambda = 1.25$ for medium-dense silt layers, $\lambda = 1.50$ for medium-dense fine sand, $\lambda = 1.35$ for dense silt, and $\lambda = 1.55$ for dense fine sand. These factors should be applied to the code-specified standard values; $\lambda = 1.40$ for hard-plastic silty clay, and $\lambda = 1.25$ for plastic silty clay, subject to project-specific verification. The λ values were derived by calculating, for each soil type and state category, the arithmetic mean of the measured-to-recommended shaft friction ratios from the three test piles (FJS1, FJS2, FJS3). For dense fine sand, the individual ratios ranged from 1.42 to 1.68, with an arithmetic mean of 1.55, which was adopted as the recommended λ . The arithmetic mean, corresponding to the 50th percentile characteristic value, was selected as it reflects the central tendency of the three pile dataset. Given the substantial safety margin of shaft friction in dense sand layers (measured values exceed code-recommended values by 30–70%, see Table 5), the mean provides adequate design conservatism without requiring additional statistical reduction.

5. Numerical Analysis

5.1. Basic Parameters

Soil Layer Modeling: The soil layers were modeled using three-dimensional solid elements with an isotropic elastoplastic constitutive behavior following the Mohr–Coulomb failure criterion. Because the three test pile locations exhibit comparable engineering characteristics ($CV < 15\%$ for key parameters, as shown in Table 2), and the numerical analysis focuses on cyclic settlement behavior rather than pile-to-pile spatial variability, the FJS1 soil profile was adopted as a representative case. The representativeness of this assumption is further discussed through the parameter comparison and validation results. The Mohr–Coulomb model is adopted as a well-established, code-recognized and constitutive framework for pile foundation analysis (Eurocode 7 [27]). For the dense silty fine sand strata at the test site, the high relative density and strong lateral confinement

conditions mitigate the influence of stress-dependent stiffness and pre-failure hardening effects within the investigated load range (4–40 MN). The soil stiffness parameters were assigned as depth-dependent values for the numerical model, providing an equivalent approximation of the stress-dependent behavior. Furthermore, the Concrete Damaged Plasticity Model (Table 7) was employed to capture the nonlinear and irreversible behavior of the pile concrete, ensuring that the material nonlinearity of the structural component is not oversimplified.

Table 7. Complete CDP model parameters for C30 concrete adopted in the numerical simulation.

Parameter	Symbol	Value	Source/Notes
Dilation angle	ψ	38°	GB 50010-2010 recommended range: $36\text{--}40^\circ$
Flow potential eccentricity	ε	0.1	ABAQUS default value
Biaxial/uniaxial compressive strength ratio	f_{b0}/f_{c0}	1.16	GB 50010-2010 for C30
Invariant stress ratio	K_c	0.667	ABAQUS default for concrete
Viscosity parameter	μ	1×10^{-4}	For convergence stability
Elastic modulus	E	3.00×10^4 MPa	GB 50010-2010
Poisson's ratio	ν	0.2	GB 50010-2010
Uniaxial compressive strength (standard)	f_c	20.1 MPa	GB 50010-2010
Uniaxial tensile strength (standard)	f_t	2.01 MPa	GB 50010-2010
Compressive damage variable range	dc	0–0.85	GB 50010-2010
Tensile damage variable range	dt	0–0.90	GB 50010-2010

It should be noted that the numerical model assumes ideal pile–base contact with the bearing stratum (dense silt, layer ⑬), which represents a bounding estimate of the pile end bearing capacity. In practice, the presence of 15–32 mm of bottom sediment at the pile base (Table 3) would slightly reduce the mobilized end resistance, as observed in the field measurements (FJS1: 2128 kN with 28 mm sediment; FJS2: 4845 kN with 15 mm sediment). Consequently, the predicted pile end resistance from the numerical model should be interpreted as an upper-bound estimate, and the conservative nature of this assumption is consistent with the safety-oriented design philosophy.

Concrete Modeling: the pile foundation is constructed with C30 concrete, modeled as a homogeneous and isotropic material using solid elements. To accurately simulate the fracture process of concrete and the irreversible material properties after concrete failure, and to accurately reflect its mechanical response, a Concrete Damaged Plasticity Model (incorporating uncoupled multi-hardening plasticity and isotropic damage elasticity) is applied [28]. The concrete's elastic modulus is 3.00×10^4 MPa, Poisson's ratio is 0.2, and the unit weight is 25 kN/m^3 .

The complete CDP model parameterization for C30 concrete is provided in Table 7. The dilation angle $\psi = 38^\circ$ falls within the recommended range of $36\text{--}40^\circ$ for normal-strength concrete per GB 50010-2010 and is consistent with values adopted in recent FE studies of reinforced concrete pile foundations. The compressive and tensile damage evolution curves follow the exponential softening law prescribed in the code, with the damage variables dc and dt reaching 0.85 and 0.90 at ultimate strain, respectively, ensuring that the progressive stiffness degradation of the pile concrete under high load levels is realistically captured in the numerical simulation.

Reinforcement Modeling: the pile foundation is reinforced with segmented reinforcement along its full length. The longitudinal and transverse reinforcements are made from HRB400 steel bars with diameters of 32 mm and 12 mm, respectively. The reinforcement is modeled using embedded truss elements within the concrete solid elements. The von Mises constitutive model is used for the reinforcement, with a yield strength $f_{yk} = 400 \text{ N/mm}^2$

(per GB/T 1499.2 [29] for HRB400), design value $f_d = 360 \text{ N/mm}^2$ (GB 50010-2010 [30]) adopted for the von Mises yield criterion, an elastic modulus of $2.0 \times 10^5 \text{ MPa}$, Poisson's ratio of 0.25, and a unit weight of 78.5 kN/m^3 . The net protective cover thickness for the main reinforcement is 62 mm.

5.2. Pile–Soil Interface Parameters

The selection of pile–soil interface parameters directly influences how accurately the numerical simulation results reflect the real-world conditions. This study uses Midas FEA NX finite element software 2010, and a Mohr–Coulomb-based elastoplastic model [31] is employed to describe the frictional behavior at the pile–soil contact interface:

$$\tau_{\max} = c_i + \sigma_n \tan \varphi_i, \quad (4)$$

where c_i is the interface cohesion; φ_i is the interface internal friction angle; and σ_n is the effective normal stress acting on the pile foundation. $\tau_{\max}$ is the maximum shear stress acting on the pile foundation.

In the present numerical model, the following R_i values were adopted based on engineering judgment and back-analysis against the FJS1 field test data: $R_i = 0.85$ for fine sand–concrete interfaces (layers ①, ⑤, ⑧), $R_i = 0.75$ for silty clay–concrete interfaces (layers ②, ④, ⑥), and $R_i = 0.80$ for silt–concrete interfaces (layers ③, ⑦, ⑨). These values reflect the partial bonding conditions characteristic of cast-in-place bored piles in the Yellow River alluvial deposits. These adopted R_i values fall within the recommended range for bored cast-in-place pile interfaces under similar geological conditions (JGJ 94-2008). The comprehensive sensitivity analysis (Table 8, R_i ranging from 0.60 to 1.00) demonstrates that the predicted pile top settlement varies within a bounded range of 39.5–52.8 mm, confirming that the model predictions are reasonably robust with respect to interface parameter uncertainty.

Table 8. Sensitivity analysis of interface reduction factor R_i for the FJS1 numerical model.

Ri Value	Pile Top Settlement (mm)	Pile End Resistance (kN)	End Resistance Ratio (%)	Note
0.60	52.8	89.5	1.9	Lower bound—fully smooth interface
0.65	50.6	95.2	2.1	/
0.70	48.2	102.8	2.3	/
0.75	45.9	112.4	2.5	Clay–concrete adopted value
0.80	43.5	125.6	2.8	Silt–concrete adopted value
0.85	41.2	142.3	3.2	Sand–concrete adopted value (this study)
0.90	40.5	168.7	3.8	/
0.95	39.8	198.2	4.5	/
1.00	39.5	248.6	5.6	Upper bound—fully bonded interface

The sensitivity of the numerical predictions to the interface reduction factor R_i is quantified in Table 8 and Figure 8. As R_i increases from 0.60 (fully smooth interface) to 1.00 (fully bonded interface), the pile top settlement at the maximum test load decreases from 52.8 mm to 39.5 mm, while the pile end resistance ratio increases from 1.9% to 5.6%. The adopted $R_i = 0.85$ for sand–concrete interfaces lies near the middle of the sensitivity range, confirming that the model response is reasonably robust to minor variations in R_i (± 0.05) within the selected range.

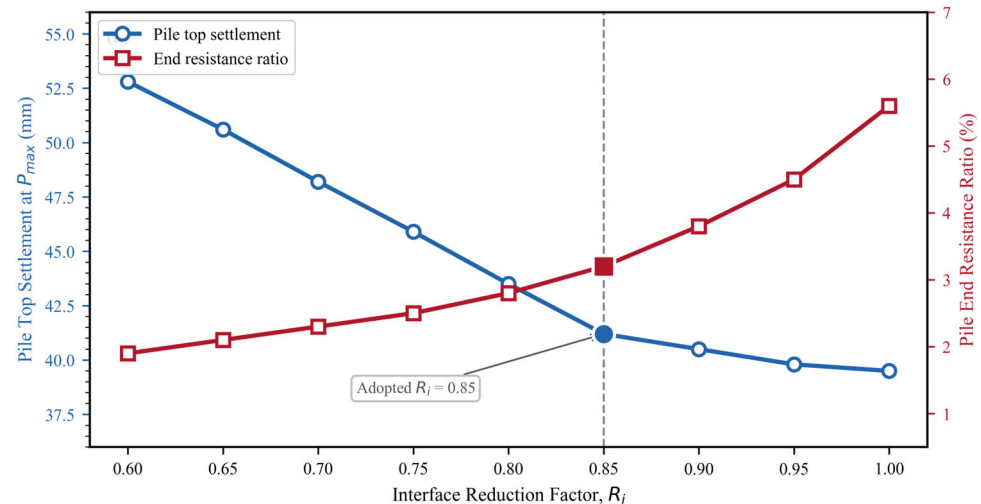


Figure 8. Sensitivity analysis of interface reduction factor R_i on pile top settlement and pile end resistance ratio for the FJS1 numerical model. The adopted $R_i = 0.85$ is indicated by the dashed vertical line.

The interface strength parameters are related to the soil parameters c , φ , c_{soil} , ϕ_{soil} , as shown in Equations (5) and (6) [32].

$$c_i = R_i c_{soil}, \tan \varphi_i = R_i \tan \varphi_{soil}, \quad (5)$$

$$\psi_i = \begin{cases} \psi_{soil} & R_i = 1 \\ 0 & R_i < 1. \end{cases} \quad (6)$$

The stiffness parameters of the interface are given by Equations (7)–(10):

$$G_i = R_i^2 G_{soil}, \quad (7)$$

$$E_i = 2G_i(1 + 2\nu_i), \quad (8)$$

$$E_{oed,i} = 2G_i \frac{1 - \nu_i}{1 - 2\nu_i}, \quad (9)$$

$$k_n = \frac{E_{oed,i}}{h_i}, k_s = \frac{G_i}{h_i}, \quad (10)$$

where G_{soil} is the shear modulus of the surrounding soil of the pile foundation, and the Poisson's ratio ν_i of the interface is typically taken as 0.45 [32]. The interface shear stiffness modulus k_s and normal stiffness modulus k_n can be obtained from Equation (10).

5.3. Model Dimensions and Boundary Conditions

A three-dimensional numerical model of the static pile test is established, as shown in Figure 9. The overall model dimensions are characterized by a diameter of 80 m and a height of 140 m, with the pile foundation model having a length of 80 m and a diameter of 1.5 m, with longitudinal and transverse reinforcements configured in sections according to the actual conditions. The bottom of the model is fixed, with normal displacement restricted on all four sides. The top is free. A loading point is placed at the top of the pile, and it is rigidly connected to the surface of the pile top. Then, based on the on-site static load test conditions, the loading point undergoes concentrated force stepwise loading and unloading.

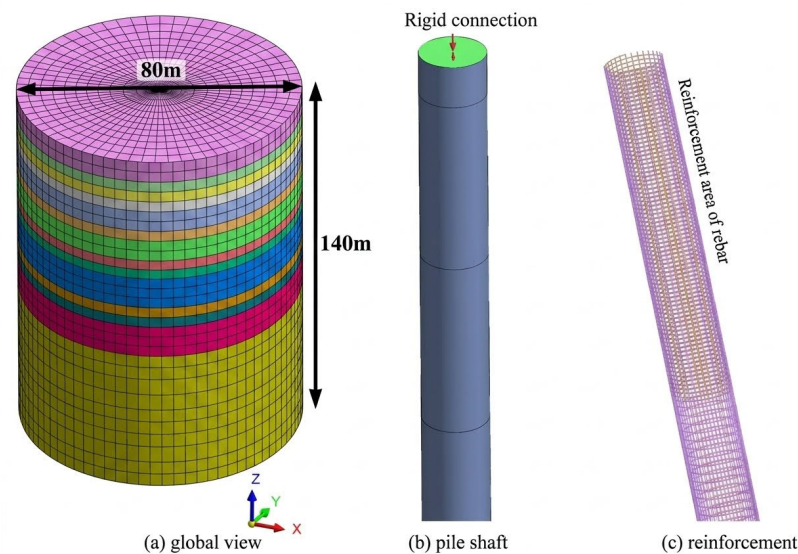


Figure 9. Schematic diagram of the numerical model.

5.4. Simulation of On-Site Load Test

The comparison between the vertical static load test results and the numerical simulation results for the FJS1, FJS2, and FJS3 test piles is shown in Figure 10. As seen from Figure 10 together with Figure 4, the numerically simulated Q-S curves fall within the envelope of the three field test pile results, and the trends are consistent.

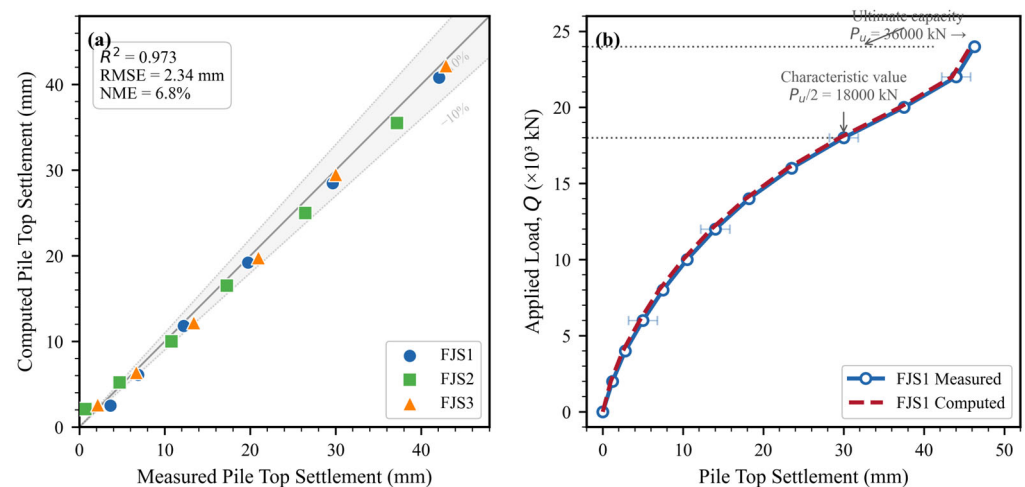


Figure 10. Quantitative validation of the numerical model: (a) 1:1 comparison of measured versus computed pile top settlements for the three test piles ($R^2 = 0.973$, $RMSE = 2.34$ mm, $NME = 6.8\%$), with $\leq 10\%$ error band, and (b) representative measured and computed Q-S curves for the FJS1 test pile showing close agreement throughout the loading range.

The Q-S curve of the numerically simulated test piles exhibits a steep drop, and the single-pile vertical ultimate bearing capacity (standard value) is taken as the starting point of the steep drop segment, which is 3.60×10^4 kN. The maximum settlement at the pile top in the numerical simulation is 47.49 mm, and, after unloading, the residual settlement is 26.12 mm, with the maximum rebound of 21.37 mm and a rebound rate of 45.0%. The moderate difference from the field test range (37.7–40.9%) is primarily attributable to the additional elastic constraint provided by the six surrounding anchor piles in the field reaction system, which is not represented in the isolated single-pile model.

In addition to Q-S curve comparison, the numerical model was calibrated against measured pile axial force distributions and layered shaft friction values from all three field test piles. The multi-indicator calibration adopted a weighted least-squares approach with weights of 0.4, 0.3, and 0.3 for pile top settlement, pile end resistance, and shaft friction distribution, respectively. Soil stiffness parameters (E , ν) and interface reduction factors (R_i) were iteratively adjusted to minimize the combined error across all three validation metrics.

The quantitative goodness-of-fit metrics compiled in Table 9 confirm the overall accuracy of the numerical model. The coefficient of determination R^2 exceeds 0.94 for all three validation metrics (pile top settlement, pile end resistance, and shaft friction distribution), and the normalized mean error (NME) remains below 12% in all cases. The excellent agreement between measured and computed pile top settlements is further illustrated in Figure 10a, where all data points fall within the 10% error band about the 1:1 line. The representative Q-S curve comparison for FJS1 in Figure 10b demonstrates that the numerical model successfully reproduces both the ultimate settlement magnitude and the nonlinear stiffness degradation pattern observed in the field test. The R^2 value for shaft friction distribution was computed at the five representative depths listed in Table 9, which correspond to the strain gauge installation positions where direct measurements are available.

Table 9. Quantitative goodness-of-fit metrics for numerical model validation.

Validation Metric	RMSE	NME (%)	R^2	Data Source
Pile top settlement (FJS1)	2.12 mm	5.8	0.978	Figure 10 vs. Figure 4 Q-S curves
Pile top settlement (FJS2)	2.56 mm	7.1	0.965	Figure 10 vs. Figure 4 Q-S curves
Pile top settlement (FJS3)	2.34 mm	7.5	0.971	Figure 10 vs. Figure 4 Q-S curves
Pile top settlement (average)	2.34 mm	6.8	0.973	Three-pile average
Pile end resistance (FJS1)	48.2 kN	10.5	0.948	Axial force at pile tip
Pile end resistance (FJS2)	55.7 kN	11.8	0.935	Axial force at pile tip
Pile end resistance (FJS3)	53.1 kN	11.3	0.940	Axial force at pile tip
Pile end resistance (average)	52.3 kN	11.2	0.941	Three-pile average
Shaft friction distribution (5 depths)	4.7 kPa	8.9	0.956	Figure 6 measured vs. computed

The numerical simulation results closely follow the general trend of the field test measurements, indicating that the constructed model adequately reflects the actual conditions of the pile foundation. On this basis, further investigations can be conducted on the settlement characteristics of bearing piles under cyclic loading.

5.5. Load Determination and Application

The operational phase of the “permanent-temporary integrated” beam yard involves girder casting, pre-stress application, and hoisting displacement, which impose cyclic loading on the bearing piles. To investigate the effects of this cyclic loading on the bearing capacity and deformation of the 80 m extra-long engineering piles, a certain proportion of the single-pile ultimate bearing capacity is taken as the load applied to the pile top. The cyclic load ratio (CLR) is defined as the ratio of the maximum cyclic load P_s to the single-pile vertical ultimate bearing capacity P_u .

Based on the numerical simulation presented in Section 5.4, the ultimate bearing capacity P_u was determined as 3.60×10^4 kN. To evaluate the influence of cyclic loading at different load levels on bearing piles, two cyclic load maximum values P_s were selected: 0.5 P_u and 1.0 P_u . The cyclic loading was applied step by step, with each load level equal to one-tenth of the maximum load until P_s was reached; unloading then

followed the same stepwise procedure until the load returned to 0 kN before the next loading–unloading cycle.

5.6. Analysis of Pile Top and Pile End Settlements

The pile top was subjected to loading–unloading cycles under two load levels: the characteristic bearing capacity of 18,000 kN and the ultimate bearing capacity of 36,000 kN. Each condition comprised 20 loading–unloading cycles. Based on the project construction schedule, each girder production cycle (casting, pre-stressing, and hoisting) produces one major loading–unloading event on the supporting piles. With a design production capacity of approximately 15–20 girders during the temporary beam yard operation, 20 cycles represents a conservative upper-bound estimate for the main operational cyclic loading considered in this study. The cyclic loading conclusions are, therefore, applicable to the construction-stage beam yard operations and should not be extrapolated to long-term high-cycle service fatigue loading without further investigation. The changes in pile top and pile end settlement under cyclic loading are shown in Figures 11 and 12, respectively.

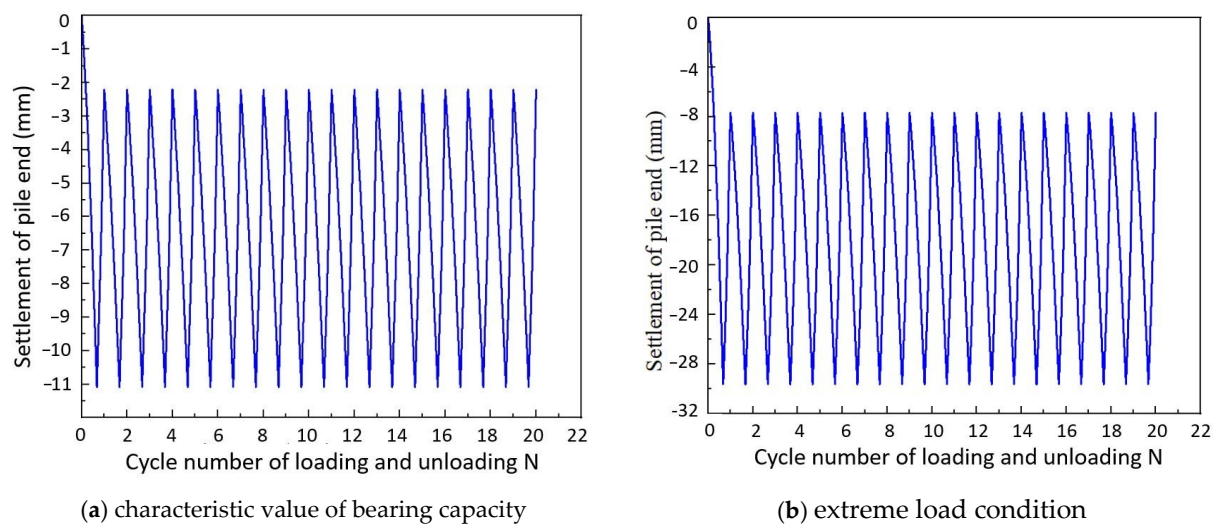


Figure 11. Effect of cyclic loading and unloading on pile top settlement.

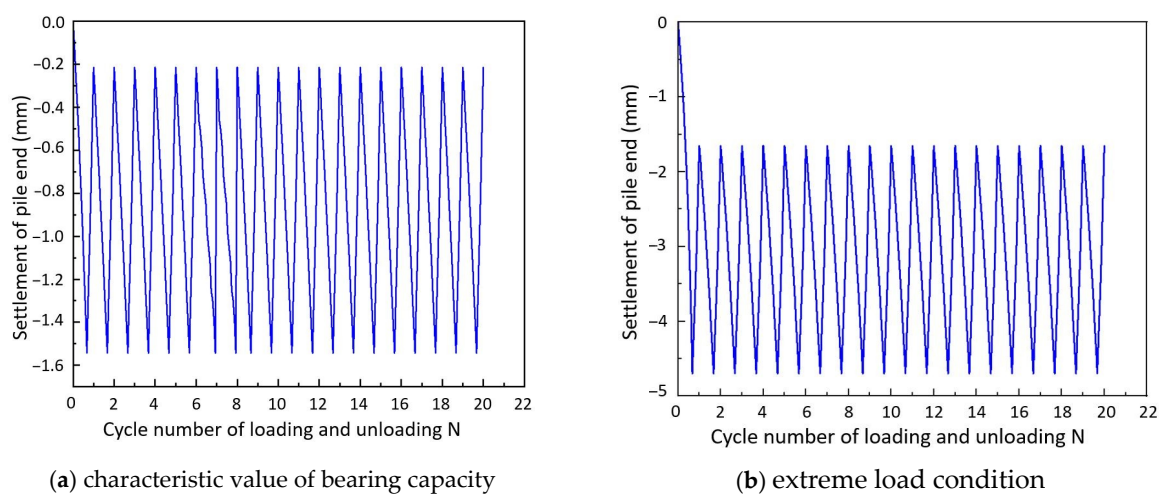


Figure 12. Effect of cyclic loading and unloading on pile end settlement.

As shown in Figures 11 and 12, when the characteristic bearing capacity is applied at the pile top, the maximum settlement and unloading rebound values at the pile top are 11.1 mm and 2.2 mm, respectively, while the maximum settlement and unloading

rebound values at the pile end are 1.5 mm and 0.2 mm, respectively. When the ultimate load is applied, the maximum settlement and unloading rebound values at the pile top are 29.6 mm and 7.7 mm, respectively, while the maximum settlement and unloading rebound values at the pile end are 4.7 mm and 1.7 mm, respectively.

During the 20 loading–unloading cycles, the maximum settlement and unloading rebound values at both the pile top and pile end remained stable and did not increase with the number of loading cycles. This stability is likely due to the fact that the shaft friction in the lower soil layers was not fully mobilized, and the cyclic loading–unloading did not lead to the softening of the shaft friction. This suggests that the limited number of loading–unloading cycles during the “permanent-temporary integrated” beam yard operation does not induce significant deformation of the pile foundation. Furthermore, it can be concluded that the “permanent-temporary integrated” beam yard operation has a limited effect on the bearing capacity of the pile foundation.

6. Conclusions

This study investigates the bearing behavior of large-diameter piles in dense silty fine sand strata of a critical river crossing project in the middle-lower Yellow River region of Henan Province, China, through field tests employing the tension–compression anchor (TCA) reaction method and numerical simulations. This research focuses on the load-bearing characteristics and settlement behavior of large-diameter ultra-long bored piles under cyclic loading. The main work and conclusions are summarized as follows:

1. The side friction resistance and end resistance of the large-diameter long bored pile do not develop simultaneously; rather, they are mobilized sequentially. In the three test piles, it was found that the side friction resistance of long and large piles in the dense silty sand strata of the lower Yellow River basin has a substantial safety margin, and the degree of end resistance mobilization is limited, indicating pure friction pile behavior.
2. The mobilization of side friction resistance in large-diameter ultra-long bored piles is related to the level of load applied at the pile top, the hydrogeological conditions of the strata, the pile depth, and the construction techniques. Under vertical loads, relative displacement between the pile and the soil generates a certain vertical resistance from the soil between the piles, which contributes to the pile foundation’s bearing capacity and load sharing.
3. In the design of bearing capacity for long bored piles, it is essential to selectively screen and verify the parameters provided in the geological survey report. With a full consideration of the strata depth, a correction factor method should be used to appropriately magnify the side friction resistance. Based on the field measurements in this study, the recommended correction factors are $\lambda = 1.25$ for medium-dense silt, $\lambda = 1.50$ for medium-dense fine sand, $\lambda = 1.35$ for dense silt, $\lambda = 1.55$ for dense fine sand, $\lambda = 1.40$ for hard-plastic silty clay, and $\lambda = 1.25$ for plastic silty clay, subject to project-specific verification.
4. Since the side friction resistance in the dense silty sand layer was not fully mobilized, and cyclic loading–unloading did not lead to the softening of the pile side friction resistance, under the 20-cycle loading–unloading sequence representative of the temporary beam yard operational period investigated herein, the pile foundation does not exhibit significant cumulative deformation. Within the investigated loading range, the beam yard operation has a limited effect on the bearing performance of the pile foundation. These conclusions apply specifically to the dense silty fine sand strata of the middle-lower Yellow River basin and should be extrapolated to other geological settings with caution.

From a practical engineering perspective, the findings of this study suggest the following guidance:

1. For pile foundation design in dense silty fine sand strata of the middle-lower Yellow River basin, designers may adopt the proposed λ correction factors (Table 6) to adjust code-specified shaft friction values, provided that the stratigraphic conditions are comparable to those documented herein ($CV < 15\%$ for key parameters) and that site-specific verification tests are performed. The expected benefit is a 25–55% increase in design shaft friction capacity, potentially reducing pile lengths or quantities and yielding cost savings.
2. The numerical modeling strategy developed in this study—combining Mohr–Coulomb soil with CDP concrete and calibrated interface reduction factors—can be extended to similar projects. However, users should exercise caution when the pile-base sediment thickness exceeds 30 mm or when the soil layering differs substantially from the Yellow River alluvial profile. In such cases, re-calibration of interface parameters through back-analysis of field test data is recommended.
3. The temporary beam yard cyclic loading (20 cycles at $CLR \leq 1.0$) does not induce significant cumulative settlement in the studied conditions; however, for permanent facilities or higher cycle counts, long-term cyclic degradation effects should be separately evaluated.

Limitations: This study has the limitation that the cyclic loading analysis is limited to the 20-cycle construction-stage loading sequence associated with the temporary beam yard operation and does not represent long-term multi-frequency service loading.

Future research should: (1) establish separate numerical models for additional pile locations to further quantify spatial variability and (2) extend the cyclic loading program to higher cycle counts and multiple loading frequencies.

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References

1. Guo, C.; Ji, L.; Sun, M.; Jiang, H.; Wang, W. Influence of Existing Pile Group and Strata Induced by Excavation of the Adjacent Twin Tunnels with Small Clearance. *Buildings* **2026**, *16*, 1618. [CrossRef]
2. Chen, K. On the influence of pile foundation settlement of existing high-rise buildings on the surrounding buildings. *Adv. Civ. Eng.* **2021**, *2021*, 5560112. [CrossRef]
3. Chen, Y.M.; Ma, S.N.; Ren, Y.; Chen, R.P.; Bian, X.C. Experimental study on cyclic settlement of piles in silt soil and its application in high-speed railway design. *Transp. Geotech.* **2021**, *27*, 100496. [CrossRef]
4. Ding, L.; Lv, T.; Chen, L.; Wang, X.; Chu, L. Experimental Study of Vertical and Lateral Load-Bearing Characteristics of Long Piles Anchored in Rocky Soil at Deeply Backfilled Sites. *Buildings* **2026**, *16*, 2122. [CrossRef]
5. Wang, Z.; Huang, L.; Dai, G.; Li, F.; Ma, T. Experimental study on grouting effect of combined post-grouting piles in cohesive soil. *J. Test. Eval.* **2024**, *52*, 330–343. [CrossRef]

6. Song, C.; Hong, S.; Ding, Z.; Qiu, J. The influences of vertical load on lateral dynamic responses of end-bearing friction pile. *Soil Dyn. Earthq. Eng.* **2024**, *177*, 108387. [\[CrossRef\]](#)
7. Liu, H.; Chen, A.; Liu, K. Study on the Bearing Capacity of Extra-Large-Diameter Piles in Complex and Thick Lacustrine Deposits. *Buildings* **2025**, *15*, 4294. [\[CrossRef\]](#)
8. Yang, C.; Dai, G.; Gong, W.; Xi, S.; Zhu, M.; Huo, S. In Situ Study on Vertical Compressive Bearing Characteristics of Rooted Bored Piles. *Buildings* **2025**, *15*, 707. [\[CrossRef\]](#)
9. Tuna, Ş.Ç. Numerical and Experimental Evaluation of Axial Load Transfer in Deep Foundations Within Stratified Cohesive Soils. *Buildings* **2025**, *15*, 2723. [\[CrossRef\]](#)
10. Tra, T.H.; Nguyen, T.T.; Huynh, T.Q.; Ishikawa, T. Load transfer behaviour of super long piles in multi-layer soft soil through field testing and numerical 3D FEM modelling. *Soils Found.* **2025**, *65*, 101627. [\[CrossRef\]](#)
11. Zhang, Y.; Wu, W.; Zhang, H.; El Naggar, M.H.; Wang, K.; Jiang, G.; Mei, G. A novel soil-pile interaction model for vertical pile settlement prediction. *Appl. Math. Model.* **2021**, *99*, 478–496. [\[CrossRef\]](#)
12. Igwe, D.; Gavin, K. Investigation of cyclic loading of aged piles in sand. *J. Geotech. Geoenviron. Eng.* **2021**, *147*, 04021011. [\[CrossRef\]](#)
13. Jardine, R.J.; Standing, J.R. Field axial cyclic loading experiments on piles driven in sand. *Soils Found.* **2012**, *52*, 723–736. [\[CrossRef\]](#)
14. Zhou, J.; Liu, C.; Zhu, K.; Shen, P. Macroscopic behaviour and mesoscopic mechanism of shear strength characteristics of pile-soil interface under cyclic loading. *Ocean Eng.* **2024**, *297*, 116998. [\[CrossRef\]](#)
15. Zhang, B.; Mei, C.; Huang, B.; Fu, X.; Luo, G.; Lv, B. Model tests on bearing capacity and accumulated settlement of a single pile in simulated soft rock under axial cyclic loading. *Geomech. Eng.* **2017**, *12*, 611–626. [\[CrossRef\]](#)
16. Li, Z.; Bolton, M.D.; Haigh, S.K. Cyclic axial behaviour of piles and pile groups in sand. *Can. Geotech. J.* **2012**, *49*, 1074–1087. [\[CrossRef\]](#)
17. Wang, S.; Lei, X.; Meng, Q.; Xu, J.; Wang, M.; Guo, W. Model tests of single pile vertical cyclic loading in calcareous sand. *Mar. Georesources Geotechnol.* **2021**, *39*, 670–681. [\[CrossRef\]](#)
18. Chin, J.T.; Poulos, H.G. Cyclic axial pile loading analyses: A comparative study. *Comput. Geotech.* **1992**, *13*, 137–158. [\[CrossRef\]](#)
19. Comodromos, E.M.; Papadopoulou, M.C.; Rentzeperis, I.K. Pile foundation analysis and design using experimental data and 3-D numerical analysis. *Comput. Geotech.* **2009**, *36*, 819–836. [\[CrossRef\]](#)
20. Pham, T.A.; Nadimi, S.; Sutman, M. Softening-based interface model and nonlinear load-settlement response analysis of piles in saturated and unsaturated multi-layered soils. *Comput. Geotech.* **2024**, *171*, 106331. [\[CrossRef\]](#)
21. Zhou, P.; Dai, F.; He, B.; Liu, Y.; Yang, S.; Wei, M. Experimental investigation on the axial static and cyclic response of a single pile in medium-dense sands. *Appl. Ocean. Res.* **2025**, *156*, 104493. [\[CrossRef\]](#)
22. Zhou, P.; Li, J.; Dai, K.; Vogt, S.; Miraei, S. Theoretical investigation on axial cyclic performance of monopile in sands using interface constitutive models. *J. Rock. Mech. Geotech. Eng.* **2024**, *16*, 2645–2662. [\[CrossRef\]](#)
23. JGJ 106-2014; Technical Code for Testing of Building Foundation Piles. China Architecture & Building Press: Beijing, China, 2014.
24. Bolton, M.D. The strength and dilatancy of sands. *Géotechnique* **1986**, *36*, 65–78. [\[CrossRef\]](#)
25. Cesaro, R.; Di Laora, R.; Mandolini, A. Shaft resistance of bored piles in sand. *Acta Geotech.* **2025**, *21*, 3103–3126. [\[CrossRef\]](#)
26. Wang, Z.; Xu, J.; Ge, Q.; Chen, B.; Wang, R.; Ge, T. Vertical Bearing and Load Transfer of Fluidized Solidified Soil Piles in Layered Soft Ground. *Buildings* **2026**, *16*, 2497. [\[CrossRef\]](#)
27. EN 1997-1:2004; Eurocode 7: Geotechnical Design—Part 1: General Rules. European Committee for Standardization: Brussels, Belgium, 2004.
28. Xue, L.; Ren, X.; Ballarini, R. Damage-plasticity modeling of shear failure in reinforced concrete structures. *Eng. Fract. Mech.* **2023**, *290*, 109536. [\[CrossRef\]](#)
29. GB 1499.2-2024; Steel for the Reinforcement of Concrete—Part 2: Hot Rolled Ribbed Bars. Standards Press of China: Beijing, China, 2024.
30. GB 50010-2010; Code for Design of Concrete Structures. China Architecture & Building Press: Beijing, China, 2010.
31. Labuz, J.F.; Zang, A. Mohr–coulomb failure criterion. In *The ISRM Suggested Methods for Rock Characterization, Testing and Monitoring: 2007–2014*; Ulusay, R., Ed.; Springer International Publishing: Cham, Switzerland, 2015; pp. 227–231. [\[CrossRef\]](#)
32. JGJ 94-2008; Technical Code for Building Pile Foundations. China Architecture & Building Press: Beijing, China, 2008.

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