

Article

Influence of Silt Physical Properties Under Pile Cap on Bearing Capacity of NT-CEP Pile Foundations

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Abstract

To clarify the poorly understood soil-structure interactions flanking the pile cap, this study systematically investigates the sensitivity of the New Type Concrete Expanded-Plate (NT-CEP) pile system to variations in sub-cap silt profiles, specifically moisture content (12%~16%) and dry density (80%~90% compaction degree). Mechanical results indicate that the pile cap and expanded bearing plates operate via a robust synergistic load-sharing mechanism, with plastic failure zones localized beneath these components. Within conventional physical limits, fluctuations in moisture and density trigger less than a 4% variance in the ultimate compressive capacity, demonstrating the remarkable structural resilience of the internal compensatory load-transfer path. Based on the evaluated boundary conditions, a site-specific operational envelope featuring a minimum compaction degree of 80% and a critical moisture threshold below 14% is recommended as a preliminary reference. Nevertheless, explicit mechanical limitations must be rigorously addressed: these quantitative thresholds are strictly benchmarked against the scaled model testing utilizing a specific silt thickness and pile geometric stiffness ratio. Significant deviations in these parameters are expected under three distinct boundary constraints: (1) altered multi-axial stress paths inherent to complex interbedded geologies; (2) catastrophic matric suction loss and pore pressure accumulation driven by elevated groundwater tables; and (3) severe skin friction degradation common in thixotropic soft clays. Consequently, these indicators constitute a context-specific design envelope rather than a rigid universal standard, providing a mechanics-driven baseline for the gradient optimization of advanced NT-CEP foundations while delineating required calibration paths for future full-scale field instrumentation.

Keywords: NT-CEP (new type concrete expanded pile); ANSYS finite element simulation; dry density; moisture content; bearing capacity



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1. Introduction

To achieve the national “dual carbon” goals and sustainable development strategy, engineering construction requires foundation systems with higher load-bearing capacities [1,2]. As a new type of pile foundation, the NT-CEP pile, through the installation of load-bearing discs on the pile shaft, the stress mechanism of the NT-CEP pile undergoes a fundamental change. The load-bearing plate can effectively bear and transfer part of the bending moment, making the bending moment distribution along the pile shaft more uniform and rational, and significantly reducing the risk of structural overturning. Its

technical characteristics can effectively compensate for the inherent defects of traditional constant-section cantilever piles. With its excellent compression, uplift, and overturning resistance, it better meets the requirements of foundation technology in high-quality development [3,4].

Currently, extensive research has been conducted both domestically and internationally on the performance and design of straight-hole piles [5–7]. My research group has previously conducted systematic studies on the load-bearing performance of NT-CEP piles; however, existing findings are largely based on the assumption that the physical state of the soil surrounding the pile is uniform and ideal [8–10].

In actual engineering projects, soil moisture content and dry density often exhibit significant spatial variability and dynamic changes due to factors such as climate, groundwater, and construction disturbances. These parameters not only directly determine the mechanical properties of the soil (such as strength, modulus, and compressibility) but also profoundly regulate the stress transfer efficiency at the soil-pile-soil interface and the mechanism of lateral friction resistance. Particularly for pile types such as NT-CEP piles, which rely on the bearing plate and enlarged head for load-bearing, any change in the physical state of the soil in the critical zone beneath the pile cap will directly affect the load distribution path and failure mode, thereby leading to significant deviations in design theories based on the assumption of homogeneous soil.

Therefore, a systematic study of the effects of moisture content and dry density on the bearing capacity of NT-CEP piles not only serves as a vital supplement to existing theories but also represents a crucial step in advancing the application of this pile type from “ideal homogeneous conditions” to “actual engineering conditions.” By controlling these two core physical parameters, this study elucidates the mechanism through which their variations affect the working characteristics of pile foundations. It provides more practical parameter guidelines for the design of NT-CEP piles under complex soil conditions, thereby enhancing their engineering applicability and reliability. This research holds strong practical necessity and clear scientific innovation.

2. ANSYS Finite Element Modeling

2.1. Soil Materials and Constitutive Models

2.1.1. Material Selection

Guided by actual engineering conditions [11,12], HRB400 hot-rolled reinforcing steel and C30 concrete are adopted as the primary materials in this study. Based on previous findings from our research group concerning the cap-soil influence range, the subsoil model beneath the pile cap is configured with a 3-m-thick layer of typical silt, followed by an underlying clay layer [13,14].

2.1.2. Soil Constitutive Models

Based on our research group’s previous model tests on NT-CEP piles [15], an analysis of three common constitutive models revealed that while the Mohr-Coulomb model is widely used, it cannot describe the behavior of materials under complex triaxial stress conditions, whereas the Cam-Clay model is primarily applicable to the compressibility behavior of soft soils. Since the Drucker-Prager (DP) yield criterion accounts for the effects of intermediate principal stresses and hydrostatic pressure, the NT-CEP finite element model established using the DP yield criterion can accurately simulate pile-soil interaction and soil failure modes in cohesive soils. Consequently, the DP yield criterion was selected for this simulation [16,17].

Matrix-based analytical characterization of plastic yielding in the soil beneath the bearing plate. To quantitatively define, from the perspective of continuum mechanics, the

critical boundary for the evolution of the soil beneath the bearing plate from a linearly elastic state to plastic shear failure, this paper introduces a generalized three-dimensional stress-yield criterion that accounts for confining pressure effects. Its analytical expression, after vectorization and matrix reorganization in three-dimensional stress space, is shown below:

$$F = \frac{6\sin\varphi}{3 - \sin\varphi} \sigma_m + \left[\frac{1}{2} S^T [M] S \right]^{\frac{1}{2}} - \frac{6c\cos\varphi}{3 - \sin\varphi} = 0 \quad (1)$$

σ_m is the average principal stress acting on the soil element (i.e., the hydrostatic pressure term, $\sigma_m = 1/3(\sigma_1 + \sigma_2 + \sigma_3)$), which is used to map the confining pressure dependence of the soil modulus and strength; c is the soil's micro-cohesion, φ is the angle of internal friction, and the remaining parameters strictly adopt the quantified experimental values dynamically evolving with moisture content and dry density as presented in the paper. Furthermore, to enable direct integration of this criterion into a three-dimensional finite element nonlinear iterative algorithm, the shear stress term in the middle of the formula is represented using a matrix operator. Here, S is the vector of partial stress components, defined as $\{S\} = \{s_x \ s_y \ s_z \ \tau_{xy} \ \tau_{yz} \ \tau_{zx}\}^T$ is the symmetric transformation weight matrix, used to correct the geometric coefficients when projecting the complex stress tensor from nine-dimensional space onto six independent components. When the stress state point satisfies $F < 0$, the soil surrounding the pile is in a stable elastic state; however, when the calculated stress trajectory touches the yield surface at $F = 0$, local shear slip lines begin to form in the soil beneath the bearing plate, ultimately inducing the propagation of compensatory loads into deeper layers.

The DP model requires two parameters—the angle of friction and cohesion—to define the soil's shear strength. Effect of moisture content on the shear strength of silt: As shown in Figure 1, an increase in the moisture content of silt beyond the optimum moisture content—similar to silty clay—weakens the interparticle attraction and interlocking action, thereby reducing cohesion. However, since silt contains a higher proportion of fine particles, interparticle contact is primarily between fine particles, and the internal friction angle is largely determined by the soil's particle structure, size, and shape. Therefore, changes in moisture content within the normal range have little effect on the angle of internal friction of silt. Within a moisture content range of 12–16%, the angle of internal friction decreases by only 1.1°.

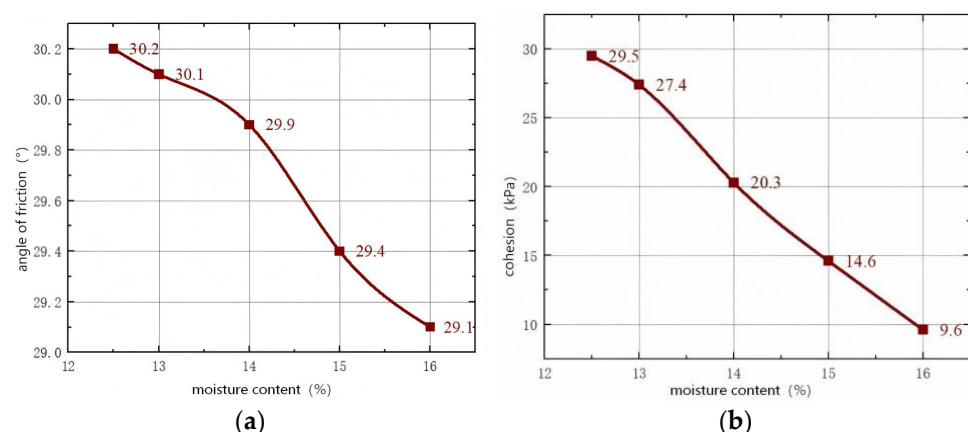


Figure 1. Shear strength parameters of silt at different moisture contents. (a) Angle of friction corresponding to different moisture contents of silt. (b) Cohesion corresponding to different moisture contents of silt.

Effect of dry density on the shear strength of silt: As shown in Figure 2, an increase in dry density results in a tighter arrangement of soil particles and enhanced interparticle

forces, thereby increasing cohesion. The higher the dry density, the more stable the soil's skeletal structure becomes, and the stronger the interparticle friction, causing the internal friction angle to increase accordingly. However, since the initial internal friction angle of silt is already quite high, the increase in dry density results in a less pronounced rise in the internal friction angle compared to silty clay; a $0.2 \text{ g}\cdot\text{cm}^{-3}$ increase in dry density leads to a 2.3° increase in the friction angle.

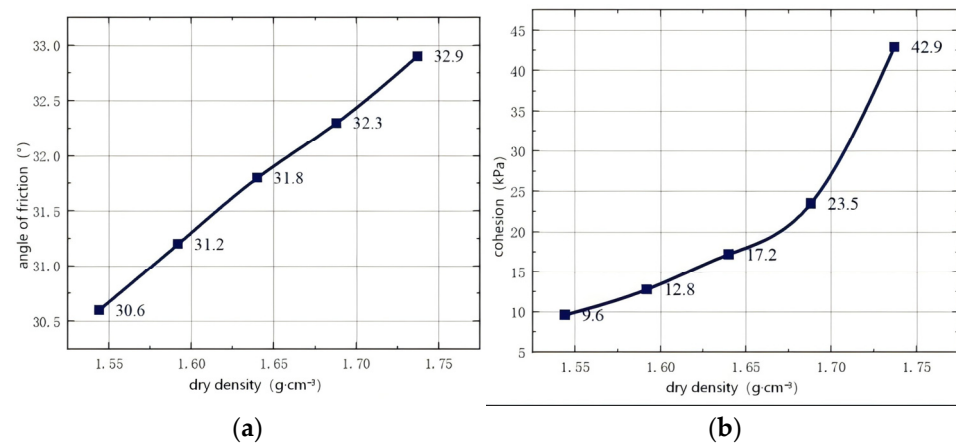


Figure 2. Shear strength values of silt corresponding to different dry densities. (a) Angles of friction corresponding to different dry densities of silt. (b) Cohesion corresponding to different dry densities of silt.

The potential impact of this constitutive limitation on the final numerical simulation results is as follows: during the initial loading stage, it can accurately predict the linear elastic stiffness and initial yield displacement of the pile foundation; however, during the deep plastic deformation stage of the full loading cycle, because the model cannot account for volumetric hardening and softening-reorganization of silty clay sand during shearing, the finite element model may slightly overestimate the lateral confinement stiffness of the soil beneath the bearing plate, leading to simulated ultimate bearing capacities that tend to be slightly higher than laboratory test values. Nevertheless, given that this error is strictly controlled within engineering tolerance limits and that the overall evolution of load transfer behavior is highly similar, the selected D-P constitutive framework has proven to be the optimal engineering solution that balances the rigor of geotechnical theory with computational convergence efficiency.

2.2. Establishment of the Geometric Model

Based on practical engineering experience [18,19], the NT-CEP pile foundation finite element model was constructed on a 1:1 scale relative to the actual project, with the soil mass defined as $10 \text{ m} \times 10 \text{ m} \times 15 \text{ m}$ (length $\times$ width $\times$ height). Figure 3 shows the NT-CEP pile engineering drawing and a schematic diagram of the drilling, reaming, and cleaning bit.

The reinforcement detailing strictly adhered to the “Technical Specifications for Building Pile Foundations” (JGJ 94-2008) and the “Code for Design of Concrete Structures” (GB 50010-2010). Specifically, the pile cap was configured with a double-layer, bi-directional reinforcement mesh ($\Phi 18@150 \text{ mm}$). The pile shaft was continuously reinforced with eight uniformly spaced longitudinal bars ($\Phi 18$) running its entire length, supplemented by transverse spiral stirrups ($\Phi 8$) at a pitch of 300 mm. The geometric parameters of the pile shaft and the schematic of the pile-soil model are illustrated in Table 1 and Figure 4, respectively. Based on these baseline configurations, ten distinct analytical models were established to evaluate the effects of varying moisture contents and dry densities. The

detailed parametric conditions are summarized in Table 2. Finally, incremental loading was applied up to the point of structural failure to capture and analyze the mechanical responses of the system.

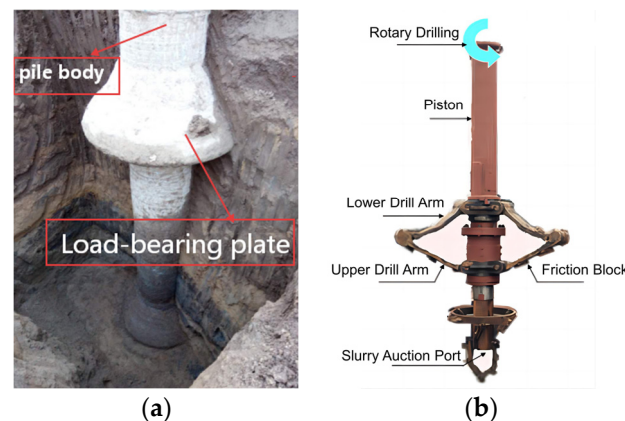


Figure 3. NT-CEP pile engineering drawings and construction process diagrams. (a) Engineering drawing of NT-CEP pile. (b) Schematic diagram of the drill bit for a drilling-reaming-cleaning machine.

Table 1. Pile shaft dimensions parameter.

Pile Length/(mm)	Pile Diameter/(mm)	Header Cantilever Diameter/(mm)	Header Slope Angle (Up/Down)/(°)	Cap Dimensions/(mm)	Cap Height/(mm)
10,000	500	750	45°/27°	2600 × 2600	800

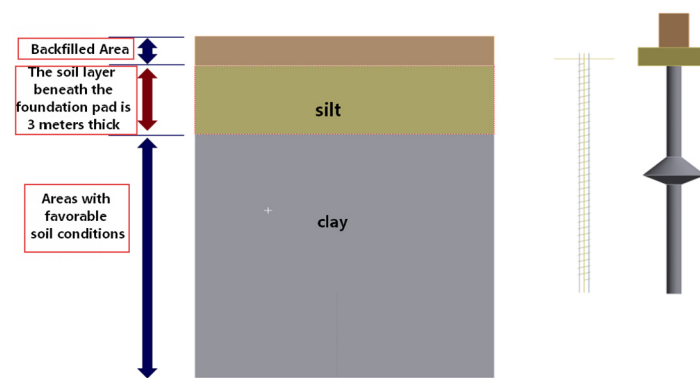


Figure 4. Schematic diagram of the pile-soil model.

Table 2. Model grouping for silty clay.

Soil Type			Silt		
Model Name	RW1	RW2	RW3	RW4	RW5
Moisture Content (%)	12	13	14	15	16
Model Name	RD1	RD2	RD3	RD4	RD5
Dry Density (g·cm ⁻³)	1.52	1.57	1.62	1.68	1.73

2.3. Mesh Generation

2.3.1. Geometric Mesh Generation

Given the geometric configuration of the excavation and the spatial symmetry of its support pile system, this study did not blindly construct a full-scale macroscopic model. Instead, to maximize computational efficiency while ensuring numerical convergence

accuracy, a typical symmetric section of the excavation system (a partial excavation model) was selected as the core computational domain.

On this basis, strict normal symmetry constraints were applied to all symmetric cross-sections. From the perspective of continuum mechanics, these symmetric boundary conditions enable the lossless and equivalent mapping of local element responses to the global stress bubble of the excavation pit, thereby perfectly reproducing the pile-soil interaction characteristics and the development of the soil's plastic zone under the same load combinations as in the full-scale excavation pit model.

For the pile and soil models, a multi-region partitioning method was employed to subdivide along characteristic edges. To balance computational accuracy and speed, hexahedral meshes were prioritized to achieve an optimal trade-off between precision and efficiency. Since the excavation depth in the simulation exceeds ten times the pile diameter, boundary effects can be neglected; however, local mesh refinement was applied to the high-stress gradient zones around the bearing plate to enhance computational accuracy. To eliminate numerical artifacts caused by spatial discretization and ensure that the simulated pile-soil mechanical responses exhibit strict grid independence, this study conducted a mesh convergence analysis on the reference model. This study planned three independent mesh density schemes, defined as sparse mesh (mesh size 500 mm), medium mesh (mesh size 200 mm), and fine mesh (mesh size 100 mm). When the mesh density was further doubled to the fine scheme, the overall load-settlement response curves almost completely coincided; consequently, the medium mesh was ultimately selected for the finite element simulation. The final mesh quality was satisfactory: the maximum aspect ratio was 12.2 (allowable value: 1000), the minimum element mass was 0.21 (lower limit: 0.05), and the average element mass was 0.88, all of which met the computational requirements. The meshing results are shown in Figure 5.

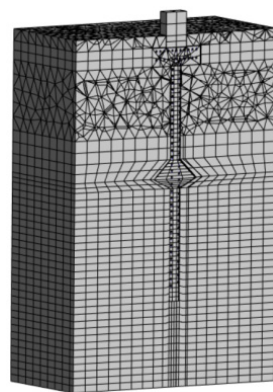


Figure 5. Grid division of the model.

2.3.2. Element Selection

In this study, the numerical modeling was conducted using the ANSYS version 2022R2 finite element package, which is headquartered in Pittsburgh, Pennsylvania, U.S. The soil domain was discretized utilizing SOLID186, a 20-node higher-order 3D solid element. The quadratic displacement shape functions inherent to these elements facilitate a highly accurate representation of the complex nonlinear deformation characteristics of the soil [20]. Furthermore, the reinforcing bars were modeled employing BEAM188 elements. Founded on Timoshenko beam theory, these elements inherently account for shear deformation effects, making them exceptionally suitable for accommodating both linear and nonlinear large-deformation analyses [21].

2.3.3. Contact Settings

The pile-soil contact is set as frictional contact with a friction coefficient $\mu = 0.3$ [22], using the penalty function method. To account for differences in elastic modulus, the pile is defined as a rigid target surface and the soil as a flexible contact surface to prevent penetration artifacts. The contact between the reinforcing bars and the concrete is set as a fixed contact, neglecting their negligible relative slippage.

2.4. Model Constraints and Loading Scheme

To mitigate boundary effects and accurately simulate soil confinement under vertical loading, appropriate boundary conditions were applied to the soil domain. The lateral boundaries were constrained against horizontal translation while permitting free vertical movement, thereby preventing unrealistic lateral soil extrusion. The basal boundary of the soil domain was defined as fully fixed. In accordance with the “Technical Specifications for Building Pile Foundations,” the ultimate failure state of the NT-CEP foundation is defined by a cumulative pile-head settlement of 60 mm. Preliminary computational trials indicated that this critical settlement threshold is exceeded when the applied load reaches 2500 kN. Consequently, the maximum vertical load for the parametric study was capped at 2500 kN [23].

3. Small-Scale Pile Model Tests in the Laboratory

3.1. Soil Sampling

3.1.1. Design of the Soil Sampler

This research stems from the National Natural Science Foundation of China project titled “Study on the Soil Failure Mechanism and Ultimate Bearing Capacity of Multi-Disc Piles with Sliding Line Theory.” Experimental studies on concrete disc piles have primarily involved small-scale half-section pile model tests and full-scale field tests on cast-in-place concrete piles; however, there is still significant room for improvement in the testing methods and instrumentation used in previous experiments. For example, in buried soil model tests, factors such as the method of soil burial, the geotechnical parameters of the soil samples, and the stiffness of the soil-bearing container significantly influence the test results. Conventional soil sampling devices often fail to meet the required rigidity, and may even cause structural damage to the soil samples, rendering them unsuitable for experimental purposes. To address this issue, the model used in this experiment is based on a pile model derived from finite element analysis. A small-scale half-section model pile was employed, scaled down to a 1:50 ratio, and a custom-made soil sampler was used to collect undisturbed soil samples from the bearing layer to construct the test model. This study investigates the influence of base diameter and slope angle on the compressive failure mechanism of concrete flared-base piles through undisturbed soil model testing [24,25]. Figure 6 shows the design drawing of the soil sampler. By conducting small-scale undisturbed soil model tests to investigate the effects of bearing plate diameter and slope angle on the compressive failure mechanism of concrete spread-foot piles, this approach not only meets the experimental requirements for soil property parameters but also offers design flexibility, making it relatively easy to implement the proposed experimental plan. It allows for multiple sets of comparative tests on individual parameters and enables direct observation of the process by which the soil beneath the bearing plate progresses from loading to ultimate failure. The conclusions drawn from these comparative tests are more rigorous and scientific.

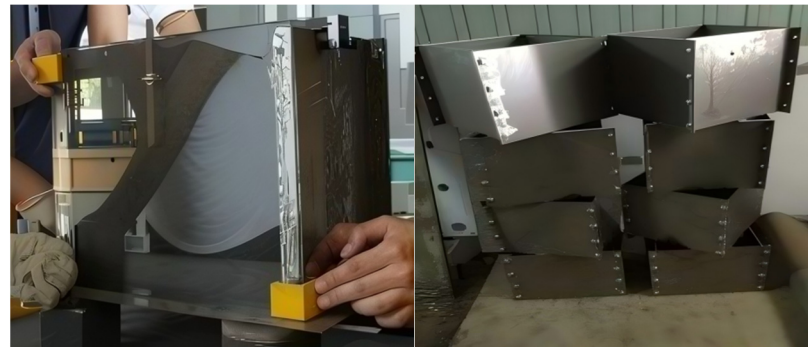


Figure 6. Actual photo of the soil sampler.

3.1.2. Soil Sampling Process

This study employed a comprehensive approach to engineering geological investigation of the site, utilizing drilling and sampling (the soil coring process is shown in Figure 7), laboratory soil tests, Standard Penetration Tests (SPT), Continuous Pile Test Unit (CPTU) tests, and shear wave velocity measurements. The maximum exploration depth reached 24 m, successfully revealing a binary macrostratigraphic configuration consisting of overlying Quaternary clay layers and underlying Cretaceous sand and mudstone layers.

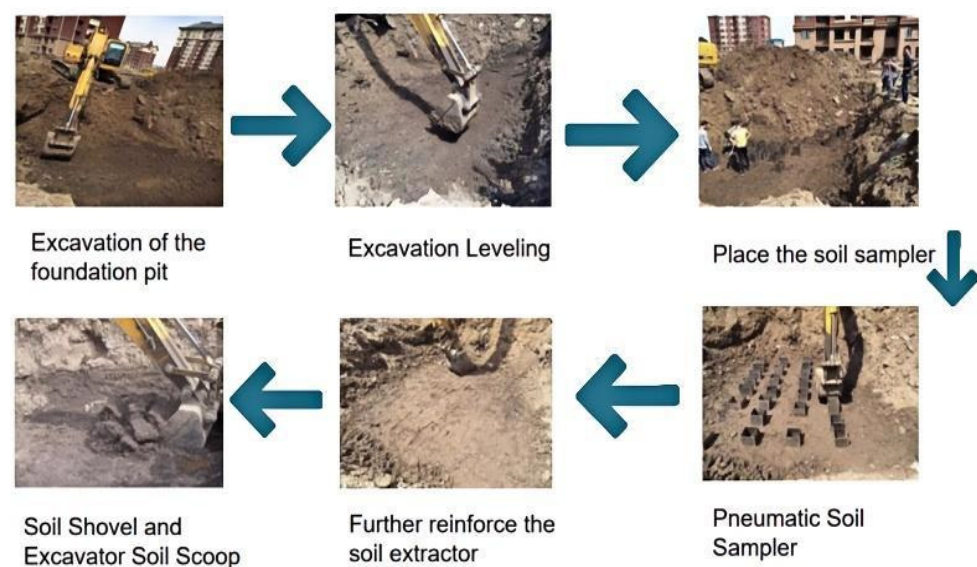


Figure 7. Soil sampling process using a soil sampler.

To ensure the physical authenticity of the boundary conditions and phenomenological parameters for subsequent numerical simulations, the core quantitative indicators from the multi-source field tests are summarized as follows: Standard Penetration Test (SPT) results indicate that the measured corrected blow count (N) of the overlying silt layer stabilizes in the range of 12–18 blows, exhibiting typical characteristics of a moderately dense skeletal structure; The Double-Bridge static cone penetration test (CPTU) revealed that cone resistance q_c exhibits a distinct step-like increase with depth (mean values ranging from 2.45 to 4.12 MPa), thereby precisely defining the spatial stiffness boundary of the critical bearing layer beneath the 3-m-thick foundation mat from a mechanical perspective; On-site shear wave velocity tests indicated that the shear wave velocity (v_s) of this composite soil layer ranged from 180 to 260 m/s, allowing the site to be classified as standard medium-soft to medium-hard construction site soil.

3.1.3. Manufacturing and Handling of Soil Samplers

Upon full extraction of the customized sampler, it was immediately and smoothly transferred to a field trimming platform for precise edge trimming and protective encapsulation (the comprehensive workflow is schematically illustrated in Figure 8). To minimize environmental disturbance to the soil skeleton structure, a putty knife was initially employed to clear the adhered overburden from the external casing, and the excess soil protruding from the cutting edges and top surface was carefully sheared off to define geometrically regular, quadrangular physical boundaries.



Figure 8. The manufacturing and transportation process of soil samplers.

Recognizing the spatial non-uniformity of the soil filling ratio inside the sampler during driving advancement, localized macro-defects or voids at both the upper and lower termination faces were systematically backfilled and compensated using intact soil excavated from the identical depth of the bearing stratum. Subsequently, a high-rigidity standardized backup plate was applied for slight volumetric compression and surface smoothing, thereby guaranteeing that the soil completely filled the internal volume of the sampler and eliminating initial boundary clearance.

To strictly preserve the natural moisture content and preclude atmospheric-induced desiccation and micro-fissuring development, the trimmed specimen ends were immediately encapsulated within multiple layers of high-density polyethylene film and hermetically sealed using high-adhesion matrix tape. This protective barrier concurrently established a robust hermetic sealing environment for moisture retention and significantly enhanced the specimen's resistance against shear disturbance and micro-vibrations during subsequent transit. Finally, the fully sealed sampler systems were systematically loaded, transported to the laboratory, and carefully deposited within a climate-controlled buffer zone for subsequent testing.

3.2. Test Piles

Following transportation to the laboratory, the embedding phase of the model pile was systematically executed prior to experimental loading. Initially, the encapsulated sealing film and matrix tape were removed, and the fastening bolts on both sides of the sampler's front face were decoupled using specialized tools to detach the steel plate. Benefiting from the symmetric structural design of the sampler, which permits dual-face alternative testing, the flatness and integrity of the exposed soil surface were rigorously inspected. Localized irregularities, if any, were meticulously remediated utilizing homogenized source soil to satisfy boundary condition requirements.

Upon verified surface compliance, a precision alignment procedure was conducted, with a distinctive centerline prominently delineated on the consolidated soil matrix. Subsequently, the half-section model pile was accurately positioned along this geometric axis and slightly pre-pressed to etch a highly definition profile of the pile architecture onto the soil. Concurrently, the spatial constraints of the bearing expanded plate were strictly regulated: the embedded depth was customized such that a 40 mm length of the pre-designed pile shaft above the uppermost expanded plate remained exposed outside the soil boundary, thereby facilitating subsequent load application and instrumentation. The comprehensive chronological sequence of this embedding process is schematically depicted in Figure 9.

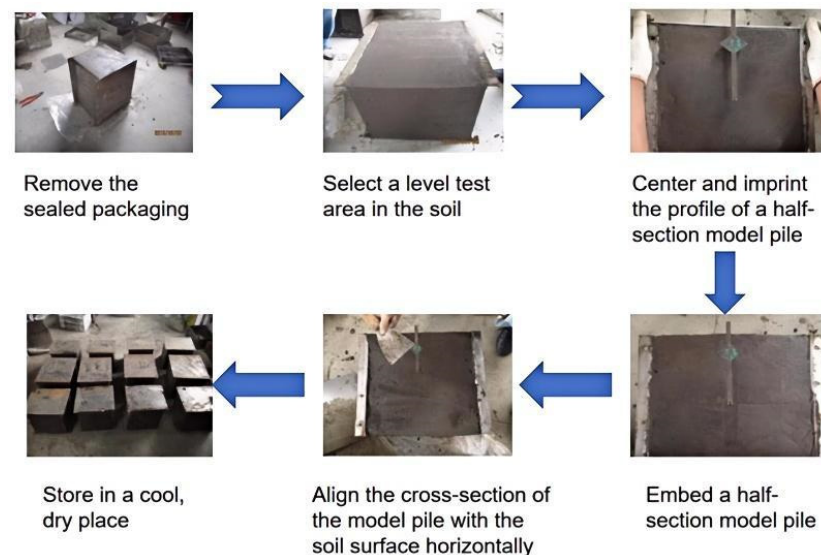


Figure 9. Process of embedding a half-section model pile into the soil sampler.

3.3. Test Loading

3.3.1. Loading Platform and Required Instruments

To accommodate the stringent boundary conditions required for the compressive failure testing of NT-CEP model piles, a specialized self-minimizing error loading framework was custom-designed (as systematically illustrated in Figure 10). The layout comprehensively incorporates the spatial dimensions of the soil sampler, the reaction force mechanism of the small-capacity hydraulic jack, and the optical clearance required for linear variable differential transformers (LVDTs). Mechanically, two heavy-duty I-beams are rigidly anchored onto a high-stiffness steel baseplate via high-strength bolts to form structural columns, between which an adjustable reaction beam is securely retrofitted. This configuration serves as a rigid backing to immovably lock the hydraulic jack and LVDTs, thereby eliminating spurious frame-compliance deformations during load advancement.

Notably, the geometric center of the platform features a hollowed-out configuration complemented by modular loading accessories. This specialized layout not only guarantees seamless execution of the present compression tests but is also intrinsically compatible with model pile pull-out (uplift) capacity investigations. Such multi-functional versatility greatly enhances the testing efficiency and provides a robust, adaptable hardware paradigm for subsequent expansions into advanced pile-soil interaction mechanisms.

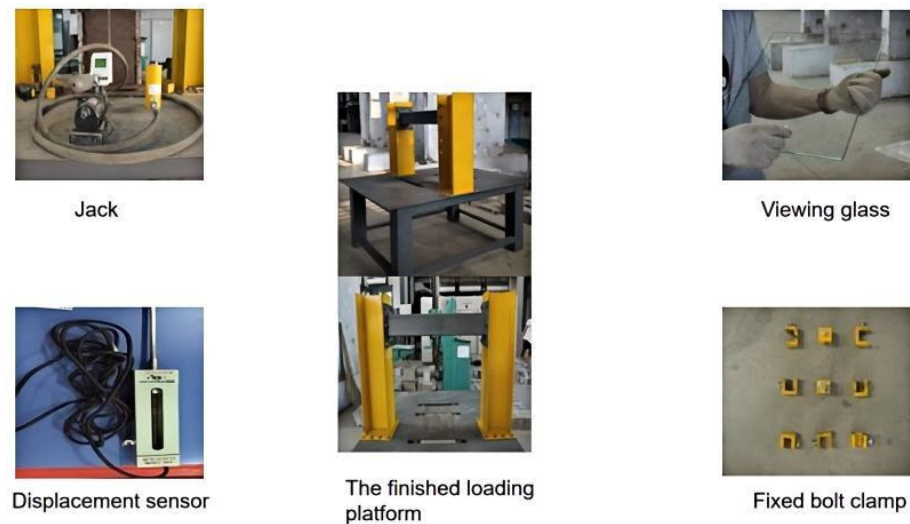


Figure 10. Loading of observation and fixed equipment.

3.3.2. Installation of the Loading Device

Following the embedding of the half-section model pile inside the sampler, the temporary assembly was cured in a climate-controlled, shady environment for a designated duration. This buffering phase was critical to facilitate the healing of the disturbed pile-soil shearing interface and achieve full mechanical compliance and stress equilibrium between the pile shaft and the soil matrix, prior to launching the loading phase.

(1) Reaction Frame Adjustment and Axial Alignment

The sampler was initially positioned onto the loading platform. Given that the structural height of the sampler (300 mm) left a geometric clearance below the reaction beam, high-rigidity spacers and backup plates were retrofitted to compensate for the height deficit, elevating the assembly into the optimal strokes of loading. The horizontal location of the sampler was systematically adjusted to ensure that the geometric centerline of the half-section model pile was perfectly coaxial with the loading axis of the reaction beam, thereby eliminating parasitic eccentric moments.

(2) Tempered Observation Glass Installation and Interfacial Clamping

A customized high-transparency tempered glass sheet, pre-cut to match the spatial dimensions of the specimen face, was aligned with the exposed testing surface. Heavy-duty bolt clamps were applied to secure the four corners of the glass onto the casing shell. During torque application, an incremental, cross-diagonal tightening sequence was strictly enforced to preclude premature brittle fracturing induced by localized stress concentrations; concurrently, the glass alignment was micro-adjusted to guarantee seamless, continuous interface contact with the soil profile, eliminating any visible air gaps.

(3) Actuator Positioning and Verticality Calibration

A hydraulic jack was subsequently suspended vertically beneath the reaction beam. The deployment was calibrated to guarantee that the primary stroke path remained strictly orthogonal to the horizontal plane throughout the progressive loading history. Based on the residual distance between the pile head and the actuator, a series of stiff steel extensions was introduced to satisfy the testing stroke requirements. The auxiliary oil pump and digital pressure indicator were integrated onto the workstation for controlled load steps and real-time readings.

(4) High-Precision LVDT Installation and Initialization

The final sequence comprised the implementation of the displacement acquisition framework. The magnetic bases of linear variable differential transformers (LVDTs) were rigidly clamped to the reaction beam. By adapting the multi-axis articulated arms, the

core tubes of the LVDTs were positioned perpendicular to the reaction beam plane, while ensuring the spring-loaded stylus tip established a point contact normal to the horizontal reference surface of the loading block. The comprehensive assembly genealogy and instrumentation layouts are systematically detailed in Figure 11.

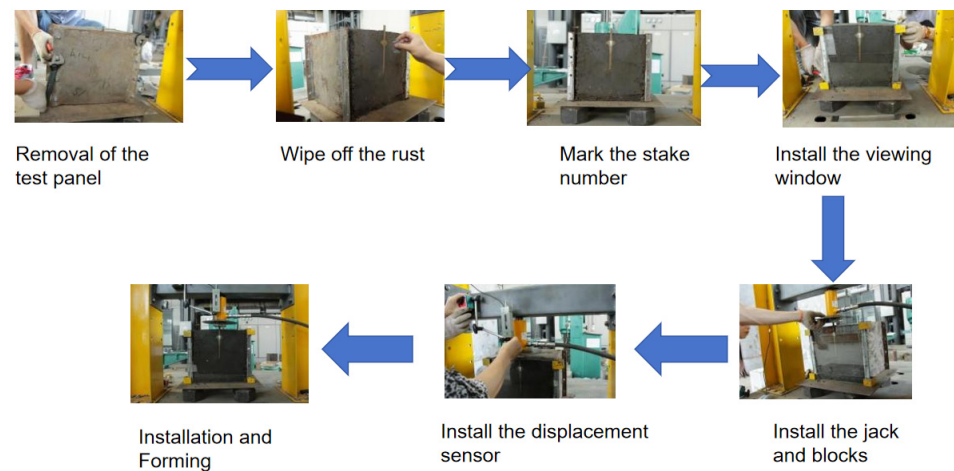


Figure 11. Installation process of the loading test equipment.

3.4. Analysis of Test Results

During the initial stage of loading, under the action of pressure, separation occurs between the pile and the soil above the bearing plate, resulting in the formation of gaps. This is caused by tensile stress in the soil above the plate, which creates cracks along both sides of the bearing plate, leading to detachment between the soil and the pile shaft. Simultaneously, cracks appear at both ends of the bearing plate due to shear failure in the soil on either side, causing the soil to slide downward; A small watermark appears on the lower part of the bearing plate. This occurs because the soil beneath the bearing plate becomes compacted under pressure, causing a sudden increase in moisture content that overflows and adheres to the observation glass. This is a visual indication of the range of influence exerted by the bearing plate's compression on the soil beneath it. A small watermark also appears in the soil directly below the base of the model pile, indicating soil compression.

As the load continues to increase, the displacement also increases; the size of the cracks on the bearing plate is a direct indication of the displacement at the top of the pile. The area of water marks in the soil beneath the bearing plate gradually expands and shows a downward converging trend. This is because the density of the soil beneath the plate continues to increase, expanding the affected area and causing the slip to converge linearly.

To facilitate comparative analysis, photographs of the scaled-down test pile before and after soil failure were compared with the results of the finite element model, as shown in Figure 12.

Comparisons show that the curves from the 1:50 indoor scaled physical tests are generally consistent with those from the finite element model at the same scale. The scientific validity and high predictive accuracy of the Druk-Plager (D-P) constitutive model for continuous media, the slip friction criterion at the pile-soil interface, and the mesh discretization topology strategy selected in this study have been fully established. The 1:1 full-scale prototype provides a solid foundation for phenomenological mechanics and methodological confidence [26].

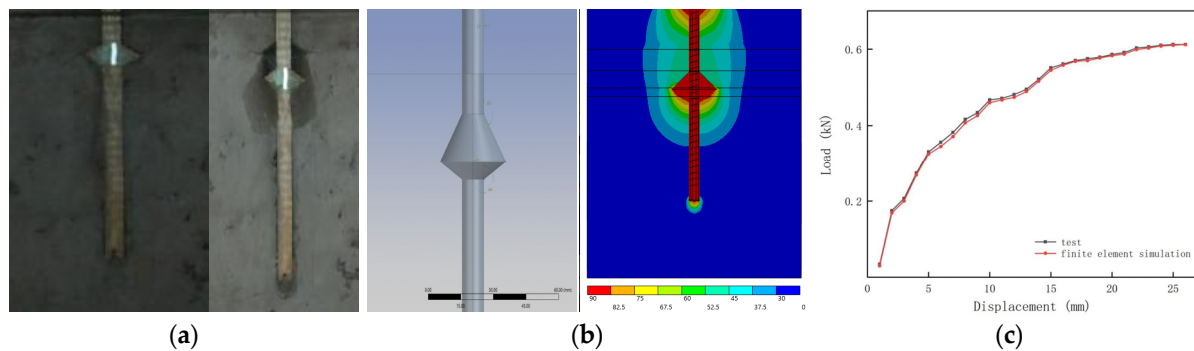


Figure 12. Comparison of experimental and simulation results. (a) Before and after the test load. (b) Before and after loading in the finite element simulation. (c) Comparison of experimental and simulation results.

Although laboratory physical model tests can accurately capture local mechanical heterogeneity, they are constrained by the physical boundaries of the laboratory, the measurement limits of loading equipment, the high cost of preparing large-scale continuous soil samples, and inherent barriers posed by scale effects, they cannot conduct full-matrix parameter sensitivity analysis at the full-scale engineering level for multi-stage spreader plate geometries (such as plate diameter ratio, plate spacing, and plate inclination angle) or for a broader range of dynamic extreme geological conditions.

In light of this, this chapter completely removes the geometric and environmental constraints of physical testing. Relying on the previously validated finite element calculation core, it comprehensively advances a large-scale, purely numerical parametric analysis of the full-scale (1:1) NT-CEP support pile system. By systematically varying different structural geometric parameters and boundary load combinations, this study further explores the internal “compensatory load redistribution mechanism” and stress bubble evolution behavior of this novel pile foundation under large-deformation limit states, thereby providing universal quantitative envelope references for high-level engineering design decisions.

4. Analysis of Finite Element Simulation Results

4.1. Analysis of NT-CEP Pile Head Displacement Under Different Moisture Contents of Silt

4.1.1. Analysis of Load–Displacement Curves

To evaluate the structural response, pile-head settlement data for the NT-CEP foundation were extracted under progressive superstructure loading up to a maximum of 2500 kN. The resultant load-settlement (Q-s) curves are depicted in Figure 13. These curves represent five distinct parametric cases, designated as RW1 through RW5, which correspond to subsoil moisture contents of 12%, 13%, 14%, 15%, and 16%, respectively.

Settlement Response Analysis: As depicted in Figure 13, the load-settlement (Q-s) curves for all parametric groups closely overlap and demonstrate a predominantly linear response during the initial loading phases, with a noticeable divergence emerging only after the applied load exceeds 2000 kN. The progression from RW1 to RW5 reveals that as the subsoil moisture content increases, the shear strength of the soil beneath the pile cap progressively deteriorates. Consequently, the initial stiffness of the foundation decreases slightly, leading to a marginal reduction in the overall bearing capacity of the NT-CEP pile system. Notably, at the maximum applied load of 2500 kN, the disparity in pile-head settlement between the driest (RW1) and wettest (RW5) conditions is a mere 6%. These findings demonstrate that variations within the typical moisture content range for silty clay (12–16%) exert a limited influence on the global bearing capacity of the NT-CEP foundation. Nevertheless, a comprehensive comparative analysis identifies 12% as the optimum moisture content, yielding the most favorable bearing performance.

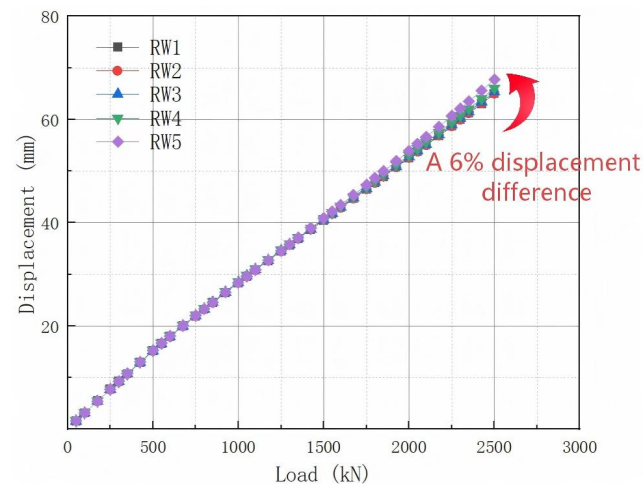


Figure 13. Displacement curve of pile top load under the influence of silt moisture content.

Influence of Moisture Content: As evidenced by Figure 13, fluctuating the moisture content of the silty clay within its typical range yields a marginal impact on the overall compressive bearing capacity of the NT-CEP pile foundation. Specifically, the maximum discrepancy in compressive capacity—observed between the boundary conditions of RW1 and RW5—is a mere 42 kN, which corresponds to a relative variance of less than 3%.

4.1.2. Analysis of Soil Displacement Cloud Maps

To further elucidate the failure mechanisms and the yield state of the surrounding soil mass, vertical (Z-axis) displacement contours were extracted for detailed investigation. Given the previously established finding that moisture content variations within the typical range for silt exert a marginal influence on the overall bearing capacity, three representative parametric cases—RW1, RW3, and RW5—were selected for this in-depth comparative analysis. To facilitate a direct and objective visual comparison, the color scales across all contour plots were strictly unified. The resulting vertical displacement fields are presented in Figure 14.

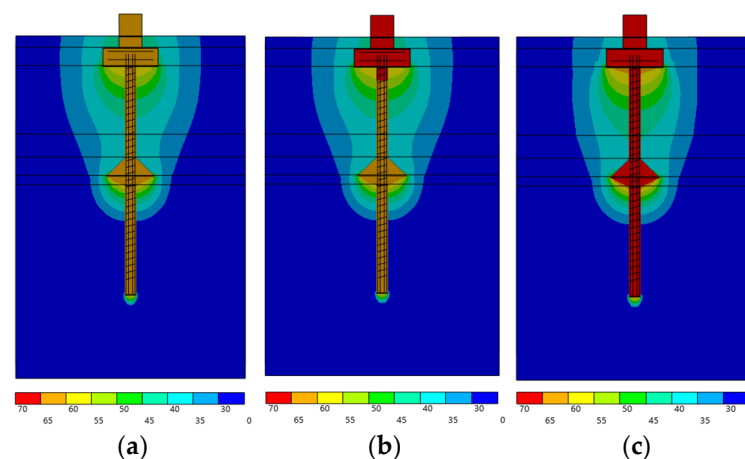


Figure 14. Z-displacement cloud map under the influence of silt moisture content. (a) RW1. (b) RW3. (c) RW5. (Units: mm).

Analysis of Soil Failure Mechanisms: As illustrated in Figure 14, the spatial distribution of displacement across all evaluated models exhibits a remarkable degree of consistency. Zones of pronounced deformation are primarily localized within the soil mass directly beneath the pile cap and the expanded bearing plate, whereas the soil surrounding the pile tip experiences comparatively minimal displacement. This distinct settlement profile

clearly dictates that, under vertical loading, the cap and the bearing plate function as the principal load-transfer components, mobilizing the majority of the bearing resistance. Furthermore, as the moisture content of the sub-cap silt fluctuates within its typical range, the global failure modes of the surrounding soil remain largely invariant. The ultimate failure state is consistently characterized by localized compressive yielding, predominantly concentrated within the strata immediately beneath the cap and the expanded plate.

Detailed Influence of Moisture Content: As illustrated in Figure 14a–c, under the ultimate applied load of 2500 kN, progressive increases in the moisture content of the sub-cap silt correspond to magnified displacement magnitudes in the soil zones directly beneath both the pile cap and the expanded bearing plate. Notably, while the vertical (Z-axis) influence depth of soil settlement beneath the cap noticeably expands, the displacement extent beneath the bearing plate remains relatively stable. Crucially, as the moisture content surpasses the optimum threshold, resulting in shear strength degradation of the silt, the lateral (X-axis) extent of soil deformation remains largely invariant. This constrained lateral displacement indicates that the load-diffusion mechanism of the sub-cap soil—its capacity to dissipate applied stress to adjacent strata—is not significantly compromised by the increased moisture. Consequently, the sub-cap soil retains a robust load-bearing capability. Furthermore, given that the load-sharing proportion of the expanded bearing plate is comparatively subordinate, the inevitable decrease in soil shear strength beneath the plate does not trigger a dramatic loss of stability. Therefore, the NT-CEP pile foundation as a whole sustains a high level of ultimate bearing capacity despite these localized moisture variations.

It is worth noting that the significant stability of the overall bearing capacity observed in this study with respect to variations in soil properties is highly coupled with the geometric and stratigraphic boundary conditions of this study. Specifically, the 3-m-thick silt layer beneath the foundation pad evaluated in this study limits the spatial expansion of the plastic slip surface induced by the foundation pad, thereby enabling the lower spread foot to effectively intercept and redistribute the transferred load. Therefore, quantitative conclusions regarding the limited overall sensitivity should be primarily based on the stratigraphic configuration involved in this study and the range of conventional geotechnical parameters examined.

4.2. Analysis of NT-CEP Pile Head Displacement Under Different Dry Densities of Silt

4.2.1. Analysis of Load–Displacement Curves

Influence of Dry Density on Bearing Performance: To evaluate the effect of soil compaction, pile-head load-settlement data for the NT-CEP foundation were extracted under a progressive superstructure load up to 2500 kN, as summarized in Table 2. Parametric analysis of the sub-cap silt indicates that an increase in dry density from $1.52 \text{ g}\cdot\text{cm}^{-3}$ to $1.72 \text{ g}\cdot\text{cm}^{-3}$ triggers a substantial enhancement in soil shear strength, evidenced by a significant 33.3 kPa increase in cohesion. The resultant load-settlement (Q-s) responses of the NT-CEP pile foundation under these varying dry density conditions are plotted in Figure 15.

Settlement Response Analysis: As depicted in Figure 15, the load-settlement (Q-s) curves across all density groups largely coincide, demonstrating a predominantly linear response during the initial loading phases. Noticeable divergence emerges only after the applied load surpasses 2000 kN. The progression from RD1 to RD5 indicates that as the dry density increases, the shear strength of the sub-cap soil improves accordingly. Consequently, the initial stiffness of the foundation increases slightly, leading to a modest enhancement in the global load-bearing capacity of the NT-CEP pile system. Under the maximum applied load of 2500 kN, the maximum discrepancy in pile-head settlement between the lowest

and highest density states (RD1 and RD5, evaluated at a constant moisture content of $W = 16\%$) reaches 5%. These findings imply that within the typical dry density range for silty clay, variations in soil compaction exert a discernible, albeit moderate, influence on the overall bearing capacity of the NT-CEP foundation. Analysis of the influence of dry density variation: When silt varies within the conventional range of dry density, the impact on the ultimate compressive bearing capacity of the NT-CEP pile foundation is minimal. The difference in compressive bearing capacity between RD1 and RD5 is 38 kN, representing a relative difference of less than 3%. In summary, when the soil layer beneath the pile cap consists of silty clay, variations in dry density within the conventional range have a minor effect on the compressive bearing capacity of the NT-CEP pile foundation. In actual engineering practice, for silty clay with a high angle of internal friction, simply meeting the conventional dry density requirements is sufficient to ensure the compressive bearing capacity of the NT-CEP pile foundation.

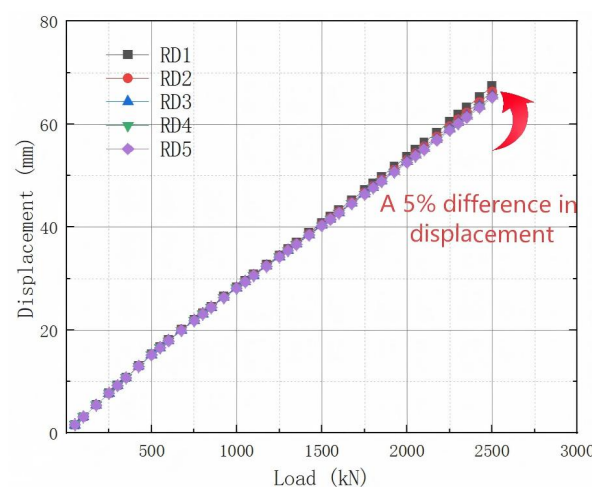


Figure 15. Displacement curve of pile top load under the influence of silt Dry Density content.

4.2.2. Analysis of Soil Displacement Cloud Maps

To gain further insight into the yielding mechanisms and spatial failure zones of the surrounding soil mass, vertical (Z-axis) displacement contours were extracted for evaluation. Acknowledging that density variations within the conventional range for silt produce a moderate, yet measurable, effect on the foundation's bearing capacity, three representative parametric models—RD1, RD3, and RD5—were systematically selected for this comparative assessment. To ensure analytical consistency and facilitate direct visual interpretation, uniform color scales were applied across all generated contour plots. The corresponding vertical displacement fields are illustrated in Figure 16.

Analysis of Soil Failure Mechanisms: As illustrated in Figure 16, the spatial displacement profiles of the sub-cap silty soil across varying dry density conditions exhibit a highly consistent distribution. Zones of pronounced deformation are distinctly localized within the soil mass directly beneath the pile cap and adjacent to the expanded bearing plate. This kinematic response confirms that the cap and the bearing plate function synergistically as the primary load-bearing components. Furthermore, despite variations in the initial compaction (dry density) of the sub-cap silt, the overarching failure modes of the soil surrounding the NT-CEP foundation demonstrate negligible variance. The ultimate limit state is consistently characterized by localized plastic yielding, predominantly concentrated within the silty strata at the base of the cap and the soil enveloping the expanded plate.

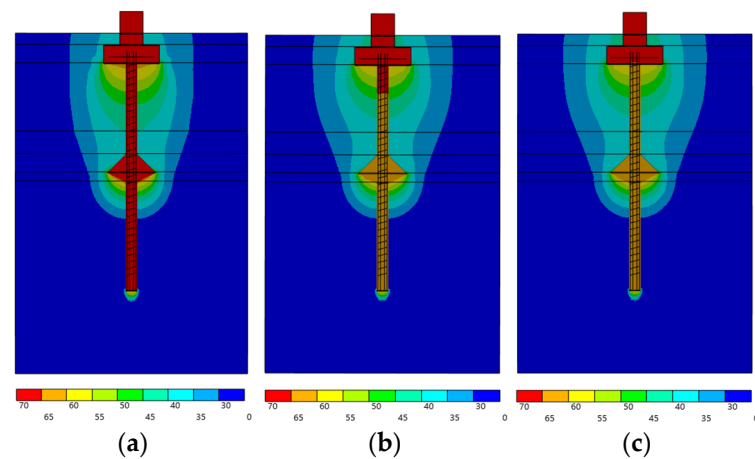


Figure 16. Z-displacement cloud map under the influence of silt dry density. (a) RD1. (b) RD3. (c) RD5. (Units: mm).

Detailed Influence of Dry Density: As depicted in Figure 16a–c, under the ultimate applied load of 2500 kN, an increase in the dry density of the sub-cap silt inherently enhances the shear strength of the soil matrix.

Consequently, the settlement magnitudes of the soil mass directly beneath both the cap and the bearing plate progressively decrease, accompanied by a slight contraction in the spatial extent of the failure zone surrounding the pile. Nevertheless, the overall variation in soil displacement remains relatively modest, and the overarching boundaries of the yield zones are largely comparable across the evaluated density states. Therefore, it can be concluded that for NT-CEP pile foundations situated with sub-cap silty strata, density fluctuations within the typical range ($1.52 \text{ g}\cdot\text{cm}^{-3}$ to $1.72 \text{ g}\cdot\text{cm}^{-3}$) exert only a minor influence on the global bearing capacity.

This observation is in strong agreement with the mechanical responses discussed in Section 4.2.1.

4.3. Comparative Analysis of Axial Stresses in Piles Under the Influence of Different Silt Layers

To further analyze the load-bearing mechanism of NT-CEP piles, axial stress contour plots were extracted for representative piles RW1, RW3, RW5, RD1, RD3, and RD5, as shown in Figure 17. For ease of analysis, the color scales in the figures have been standardized.

Analysis of Load Transfer Mechanisms: The axial stress distribution profiles along the pile shafts are highly consistent between the evaluated NT-CEP models. A pronounced discontinuity in axial stress is observed at the interface between the expanded bearing plate and the upper pile shaft. Specifically, the segment spanning from the base of the pile cap to the upper face of the bearing plate exhibits significant compressive stress concentrations, whereas the lower pile shaft beneath the expanded plate experiences substantially reduced stress levels. This phenomenon elucidates the load-path dynamics: upon application of the superstructure load, the pile cap initially absorbs a proportion of the applied force. The residual load is subsequently transferred downward through the upper pile shaft and is predominantly intercepted and dissipated by the expanded bearing plate. Consequently, this mechanism induces high compressive stresses in the upper shaft while effectively shielding the lower shaft. Ultimately, this stress distribution confirms that under vertical loading, the global bearing resistance of the NT-CEP pile foundation is primarily mobilized through the synergistic action of the pile cap and the expanded bearing plate.

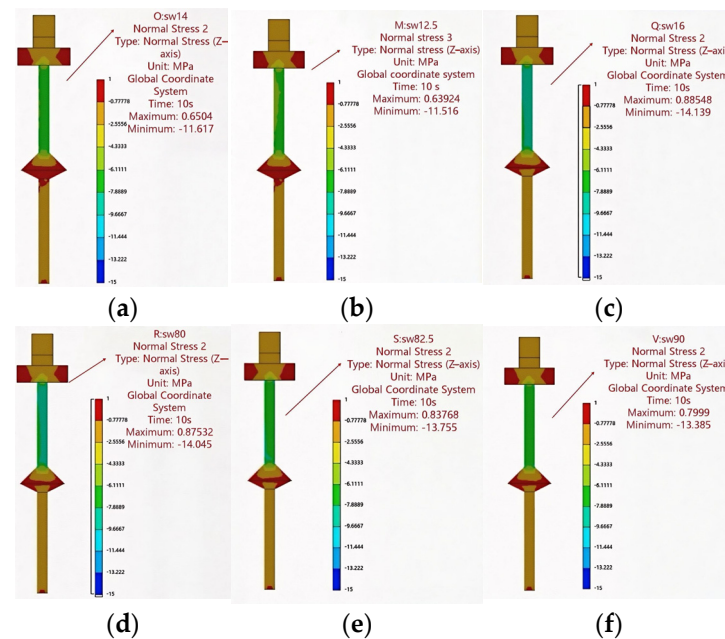


Figure 17. Axial stress cloud map of pile body under the influence of moisture content and dry density of silty clay. (a) RW1. (b) RW3. (c) RW5. (d) RD1. (e) RD3. (f) RD5.

Influence of Soil Parameters on Load Transfer: As illustrated in Figure 17a–c, the axial compressive stress within the upper pile shaft intensifies concurrently with an increase in the moisture content of the sub-cap silt. This phenomenon occurs because higher moisture content degrades the shear strength of the soil, thereby diminishing the direct bearing contribution of the sub-cap soil and reducing the load-sharing proportion of the pile cap. Consequently, to maintain equilibrium, a greater fraction of the applied load is diverted downward to the expanded bearing plate, elevating its load-sharing ratio and inducing higher axial compressive stresses within the upper shaft.

Conversely, as evidenced by Figure 17d–f, an increase in the dry density of the sub-cap silt enhances the soil’s shear strength. This improvement augments the load-sharing capacities of both the cap and the upper soil matrix, subsequently alleviating the load demand on the expanded plate and reducing the axial compressive stress in the upper pile shaft.

Synthesizing the findings from Sections 4.1 and 4.2, a critical mechanism emerges: while variations in sub-cap silty clay properties within typical ranges exert a marginal influence on the global ultimate bearing capacity, they trigger a significant internal load redistribution between the pile cap and the expanded bearing plate. A comprehensive evaluation of the axial stress fields confirms that the cap and the bearing plate act synergistically to support the majority of the applied load. Ultimately, the internal load-sharing distribution is highly sensitive to the properties of the sub-cap soil; enhanced soil shear strength directly correlates with an increased load-sharing proportion for the pile cap, demonstrating a dynamic and compensatory load-transfer behavior within the NT-CEP system.

4.4. Comparative Analysis of XZ-Directional Shear Stress Contour Plots Under the Influence of Different Silt Layer Properties

To further analyze the failure mechanism of the soil surrounding NT-CEP piles, XZ-direction shear stress contour plots were extracted and analyzed using RW1, RW3, and RW5, as well as RD1, RD3, and RD5 as representative cases. As shown in Figure 18, the color scale units in the images have been standardized for ease of analysis.

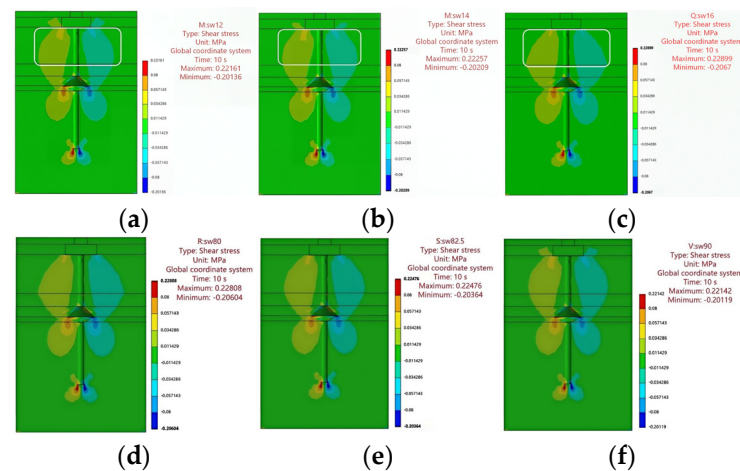


Figure 18. Cloud map of XZ shear stress of soil under the influence of moisture content and dry density of silty clay. (a) RW1. (b) RW3. (c) RW5. (d) RD1. (e) RD3. (f) RD5.

Analysis of Load-Bearing Mechanisms and Shear Stress Distribution: An evaluation of the XZ-plane shear stress fields reveals a highly consistent distribution profile between the two sets of evaluated NT-CEP models. Notably, pronounced shear stress concentration localizes at the pile tip, yielding the highest absolute magnitude of shear stress. However, owing to the significantly restricted interaction area at the pile tip, its actual contribution to the global load-sharing ratio remains disproportionately low. Conversely, the primary domains of intense stress concentration within the soil mass are predominantly localized directly beneath the expanded bearing plate. This distribution profile decisively confirms that the soil underlying the expanded plate mobilizes the overwhelming majority of the foundation’s load-bearing resistance. These observations are in complete accordance with the mechanical deductions established in the preceding analyses.

Influence of Soil Variables on Shear Stress Response: Utilizing Figure 18a–c as representative examples, it is observed that under a constant superstructure load, an increase in moisture content progressively deteriorates the shear strength of the sub-cap silt. Consequently, the fraction of the load directly supported by the silt decreases, accompanied by a marginal contraction in the spatial extent of the shear stress zones within the sub-cap soil. This phenomenon can be elucidated fundamentally: while the elevated moisture content induces a substantial degradation in soil cohesion (c), the angle of internal friction (ϕ) remains largely unaffected. Furthermore, because the soil strata immediately beneath the pile cap are subjected to substantial vertical normal stresses transmitted directly from the cap structure, this high confining pressure enables the silt to sustain robust shear resistance. As a result, the foundation experiences only a slight decline in its overall bearing capacity. Crucially, owing to the preservation of a high internal friction angle, the sub-cap soil retains its efficacy in diffusing the applied load laterally; indeed, the lateral (X-axis) distribution extent of the shear stress field within the silt actually exhibits an expansion.

Conversely, Figure 18d–f the antithetical trend. An increase in the dry density of the sub-cap silt inherently enhances the overall shear strength of the soil matrix. This improvement augments the load-sharing proportion of the pile cap and elevates the internal shear stress mobilized within the soil, thereby yielding a moderate, yet distinct, enhancement in the global bearing capacity of the NT-CEP pile foundation.

To quantitatively characterize the influence of environmental physical parameters on the evolution of load distribution among the components of the new concrete expanded-disc pile (NT-CEP), this paper extracted the load distribution ratios of each component under ultimate load conditions, as shown in Table 3. The data clearly indicate that as the soil moisture content increased from 12% to 16%, the load-sharing ratio of the cap decreased

significantly from 32.5% to 18.2% due to water-induced softening of the silt beneath the cap; In contrast, the load-sharing ratio of the lower flange structure dynamically increased from 40.2% to 53.6%, exhibiting a pronounced compensatory load-taking-over characteristic. This quantitative trend strongly supports the aforementioned mechanical interpretation of the internal load redistribution behavior (details are shown in Table 3).

Table 3. Quantitative analysis of load-sharing ratios for each component at the ultimate load-bearing state.

Test Cases	Physical Parameter	Pile Cap (%)	Pile Shaft (%)	Expanded Plate (%)	Pile Tip (%)
Fluctuations in moisture content RW = (12%, 14%, 16%)	RW1 = 12%	32.5	15.1	40.2	12.2
	RW3 = 14%	26.3	14.8	46.5	12.4
	RW5 = 16%	18.2	14.2	53.6	14.0
Fluctuations in dry density RD = (1.52 g·cm ^{−3} 1.62 g·cm ^{−3} 1.73 g·cm ^{−3})	RD1 = 1.52 g·cm ^{−3}	20.4	13.9	52.1	13.6
	RD3 = 1.62 g·cm ^{−3}	26.3	14.8	46.5	12.4

5. Conclusions

A refined three-dimensional nonlinear numerical framework incorporating complex pile-soil interaction mechanics was developed and rigorously benchmarked against scaled laboratory testing. Based on systematic parametric sweeps of NT-CEP pile configurations within silty profiles, the principal findings are synthetically formulated as follows:

(1) Spatial Failure Invariance and Component Synergism: Continuum field diagnostics demonstrate that the macroscopic shear failure modes surrounding NT-CEP foundations remain fundamentally invariant across diverse soil states and spacing configurations. Plastic yielding and pronounced strains are strictly localized in the strata immediately beneath the pile cap and enveloping the expanded plate shoulders, whereas the pile tip experiences minimal disturbance. This kinematic alignment conclusively confirms that the cap and expanded plate operate via a robust synergistic load-sharing mechanism, fundamentally altering the traditional linear load-transfer paths of constant-diameter bored piles.

(2) Parameter Sensitivity Disparity and Active Compensatory Load-Sharing: Within conventional physical ranges, fluctuations in silt moisture content 12% ~ 16% and dry density (80% ~ 90% compaction) trigger severe microstructural rearrangements of soil cohesion (c), which drops sharply from 29.5 to 9.6 kPa and climbs from 9.6 to 42.9 kPa, respectively. Crucially, this localized deterioration exerts a negligible impact on the global ultimate compressive capacity (with variations tightly bounded below 4%). This macroscopic resilience is actively driven by a spontaneous internal “compensatory load-sharing mechanism”—the instant the sub-cap interface softens, the superstructure load field is automatically diverted down the shaft and aggressively intercepted by the deeper, un-degraded geometric boundaries of the expanded plates, guaranteeing global structural stability.

(3) Design Control Envelopes and Geo-Environmental Boundary Limitations: Synthesizing field tests, current industry specifications, e.g., JGJ94-2008, and parametric simulations, a targeted construction control baseline featuring a minimum compaction degree of 80% and a critical moisture threshold below 13% is formulated. Nevertheless, explicit boundary limitations must be recognized: these quantitative envelopes are strictly tied to the specific stratigraphic thickness and geometric stiffness ratios evaluated herein. Significant deviations in these operational thresholds are expected under intricate stress paths, including overlapping stress zones in interbedded geologies, catastrophic matric suction loss under elevated groundwater tables, or thixotropic skin friction degradation. Future research incorporating multi-axial (combined vertical and horizontal) loading regimes

and full-scale field instrumentation is highly warranted to generalize and calibrate this predictive theoretical framework.

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Abbreviations

The following abbreviations are used in this manuscript:

NT-CEP piles	New Type Concrete expanded-plate piles
D-P	Drucker-Prager

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