

Article

Study on Mechanism of Soil Displacement Effect in Large-Diameter PHC Pipe Piles

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Abstract

In order to investigate the soil displacement effects and penetration resistance mechanisms of large-diameter PHC pipe piles (1200 mm) in complex railway geology, a tripartite framework combining field tests, theoretical analysis, and numerical simulations was established based on the Xiong'an–BDA Express Line project. A coupled discrete–continuum analysis using the Coupled Eulerian–Lagrangian (CEL) method was conducted to model the large-deformation process of pile driving in soft clay and stratified layers. The results indicate that the installation process induces a “squeezing effect” that critically enhances pile–soil interfacial friction. The theoretical analysis incorporating the extended Lade–Duncan yield criterion significantly improved prediction accuracy, reducing the relative error of side friction from 22% (using the Mohr–Coulomb model) to 5%. Furthermore, the CEL simulation demonstrated high reliability in predicting deep-depth friction and pile tip resistance, effectively capturing the stress redistribution in complex strata. Therefore, the combined application of pre-drilling and large-diameter piles is recommended for deformation-sensitive infrastructure, and the proposed validated framework offers practical guidance for design optimization and parameter selection in similar geological conditions.

Keywords: PHC pipe piles; pile driving; pre-drilling hole; CEL method

1. Introduction

The installation of pile foundations in saturated soft clay, whether by driving or pressing, inevitably induces significant soil displacement. This process generates excess pore water pressure around the pile shaft and substantially alters the physical–mechanical properties of the surrounding soil, such as the density, void ratio, and moisture content [1]. This “squeezing effect” is particularly critical in deformation-sensitive infrastructure projects, such as urban transit systems and high-speed railways, where uncontrolled ground movements pose a severe threat to the safety of adjacent buildings, existing railway lines, and underground pipelines. For large-diameter pre-stressed high-strength concrete (PHC) pipe piles, the large pile dimensions further intensify this squeezing effect. Consequently, balancing construction efficiency with the control of soil disturbance while maximizing shaft friction remains a central challenge in both engineering practice and theoretical research.

To gain a deeper understanding of the stress–strain behavior during pile installation, existing research has primarily relied on in situ testing, theoretical analysis, and numerical simulation. With advancements in computational mechanics, methods capable of handling



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large deformations and mesh distortion—such as the Coupled Eulerian–Lagrangian (CEL) method embedded in Abaqus—have proven reliable for geotechnical problems involving significant penetration, such as pile driving. However, current CEL-based studies (e.g., those by Qiu et al. [2], Hu et al. [3], and Tho et al. [4]) have largely focused on conventional pile sizes or idealized, simple stratified seabeds (e.g., sand overlying clay). Furthermore, theoretical analyses based on cylindrical cavity expansion theory typically employ Mohr–Coulomb or Tresca yield criteria. These criteria often fail to precisely capture the complex stress–strain behavior and non-linearity of soft clays under high-stress conditions.

In summary, to date, few studies have focused on the interaction mechanism of large-diameter pipe piles installed with pre-drilling techniques in complex geological conditions characterized by soft clay, hard interlayers, and multiple stratigraphic interfaces. Existing theoretical models require improvement to accurately simulate large deformations in soft clay, and there is a lack of systematic validation against specific large-scale engineering backgrounds.

To address these limitations, this study relies on the Xiong'an–BDA Express Line (R1 Line) project to investigate 1200 mm diameter PHC pipe piles installed using a 700 mm pre-drilling process in complex geological settings. The main objective of this study is to establish a tripartite “Field Test–Theoretical Analysis–CEL Simulation” verification framework to reveal the pile–soil interaction mechanism at both micro and macro levels. Specifically, this study aims to achieve the following:

(1) Elucidate the squeezing mechanism in complex geological conditions: Unlike previous studies restricted to homogeneous or simple bi-layered soils, this research focuses on complex railway geology containing artificial fill, soft alluvial layers, and hard interlayers. It elucidates the stress redistribution and displacement field evolution specific to the combined “large-diameter pile + pre-drilling” process.

(2) Integrate extended constitutive models to improve prediction accuracy: The extended Lade–Duncan yield criterion is introduced into the theoretical analysis of cavity expansion. By overcoming the limitations of the traditional Mohr–Coulomb model in simulating large deformations in soft clay, this approach aims to significantly improve the accuracy of pile-side friction prediction (reducing the relative error from approximately 22% to 5%).

(3) Validate the engineering applicability of numerical methods: Using real-time data from field cone penetration tests (CPTs) and static load tests, this study rigorously validates the capability of the CEL method in simulating deep-depth friction and pile tip resistance, providing a validated methodology for optimizing the design of similar large-scale projects.

2. Project Overview

The Xiong'an New Area-to-Beijing Daxing International Airport Express Line serves as a key regional rapid-transit link between Daxing District, Yongqing County, Bazhou City, and Xiong'an New Area. Spanning Beijing's Daxing District, Langfang City of Hebei Province, and Xiong'an New Area, the project involves complex environmental conditions, with tunnels under the Yongding River and Langzhuo Expressway and overpasses across multiple railways (Jingbao, Jingjiu); highways (Rongwu, Dagang); natural features such as the Daqing River; numerous villages; nurseries; and farmland. To enhance construction efficiency and ensure pile quality, large-diameter PHC precast pipe piles (external diameter: 1200 mm; internal diameter: 900 mm) were used for the foundation work. The use of such large PHC precast pipe piles is uncommon in high-speed rail projects, so research on their bearing capacity and soil displacement effects during pile installation is particularly crucial.

2.1. Engineering-Geological Profile

According to the “Detailed Geological Survey Report for the Bazhou–Yongqing Section Substation of Xiong’an New Area to Beijing Daxing International Airport Express Line Project,” the exploration area comprises four major stratigraphic units: the artificial accumulation layer (Q_4^{ml}), Quaternary Neocene sedimentary layer and Holocene alluvial–proluvial layer (Q_4^{al+pl}), Upper Pleistocene alluvial–proluvial layer (Q_3^{al+pl}), and Middle Pleistocene alluvial–proluvial layer (Q_2^{al+pl}). The soils that occur in these layers include artificial fill, Neocene deposits, and organic soil. Figure 1 shows the engineering-geological profile. The soil types identified within the stratigraphic sequence are presented in Table 1.

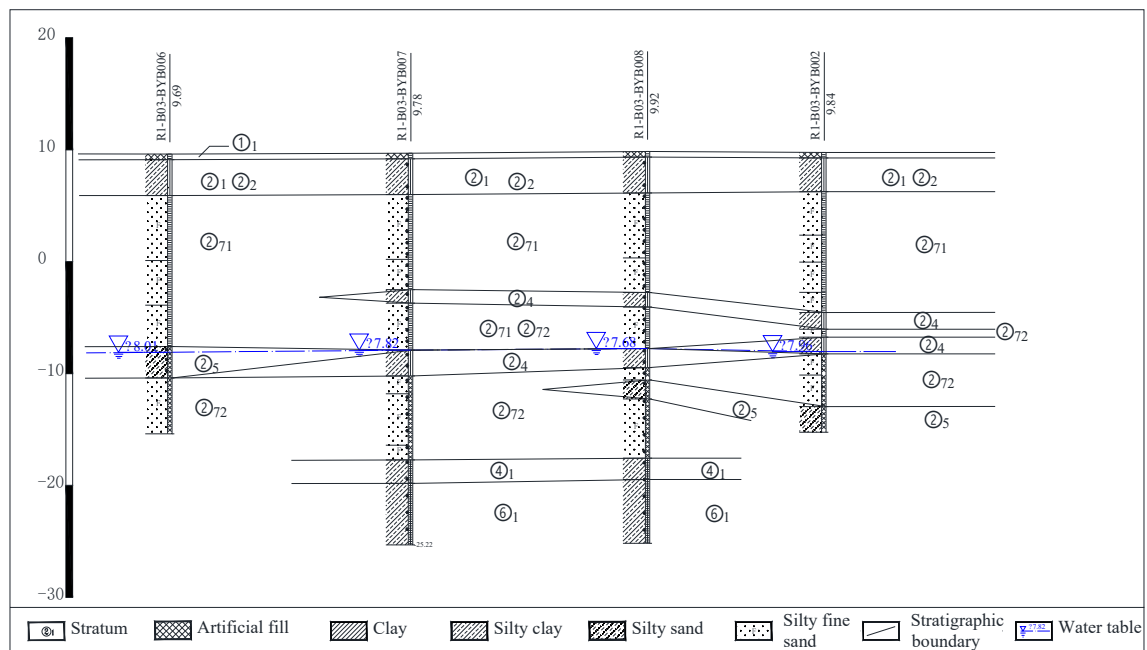


Figure 1. Section drawing of the engineering geology.

Table 1. Soil type and stratigraphic units of each stratum.

Stratum	Soil Type	Stratigraphic Units
1-1	Artificial fill	Q_4^{ml}
2-1	Silty clay	Q_4^{al+pl}
2-2	Silty clay	Q_4^{al+pl}
2-4	Silty clay	Q_4^{al+pl}
2-5	Silty sand	Q_4^{al+pl}
2-71	Silty fine sand	Q_4^{al+pl}
2-72	Silty fine sand	Q_4^{al+pl}
4-1	Silty clay	Q_3^{al+pl}
6-1	Silty clay	Q_2^{al+pl}

2.2. Pile-Driving Process

Before pile driving, pre-drilled holes with diameters of 700 mm were created at designated pile locations using a rotary drilling rig when necessary (Figure 2). After pre-drilling, the PHC pipe piles were segmentally driven into the ground using a hydraulic hammer. Since sensors were installed inside the piles, additional end plates were added at the segment joints to facilitate sensor wiring (Figures 3 and 4). Then, the pile ends were welded using CO_2 -shielded welding. After the segments had been connected, the piles were driven into the ground using the hydraulic hammer (Figure 5). Depth markers were

engraved on the pile sides (Figure 6), and the number of hammer blows per meter during pile driving was recorded.



Figure 2. Pre-drilled holes in the test area.



Figure 3. End plates for PHC pile connections.



Figure 4. Connecting sensor signal cables.



Figure 5. Hydraulic-hammer pile-driving site.



Figure 6. Depth markers on the pile sides.

Figure 7 shows the layout of the test piles, where the natural ground surface represented the coordinate origin. Piles #1 and #2 were employed to study the soil plug effect, with lengths of 20 and 28 m, respectively. Pile #1 was not pre-drilled, whereas Pile #2 was pre-drilled to 25 m. Piles #3 and #4 were employed in bearing capacity tests, with embedment lengths of 32.3 and 34.7 m, respectively; both were pre-drilled to 32 m. Table 2 lists the parameters of the test piles. In addition, nine bored cast-in-place piles were installed as anchor piles, each with an embedment length of 38 m.

Table 2. Parameters of the test piles.

Pile Number	Pile Top Coordinates (m)	Pile Base Coordinates (m)	Pile Length (m)	Pre-Drilled Depth (m)
#1	0	−20	20	0
#2	−1	−29	28	25
#3	0	−32.3	32.3	32
#4	0	−34.7	34.7	32

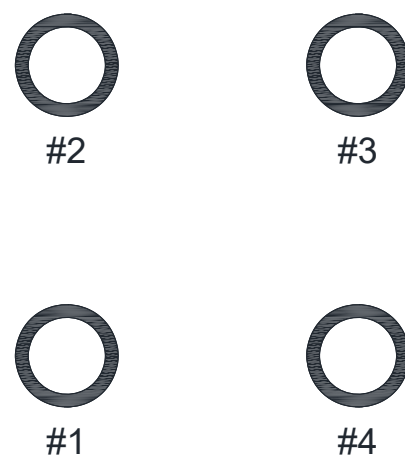


Figure 7. Plan showing the layout of the test piles.

2.3. Single-Pile-Bearing Capacity Test and Results

A vertical static load test on the PHC pipe pile was conducted using an anchor pile reaction system. Figure 8 shows a photograph of the loading conditions.



Figure 8. On-site bearing capacity test.

The test results were evaluated according to the Technical Specification for Testing of Foundation Piles in Railway Engineering (TB 10218-2019) [5]. The anchor pile method was used to observe the settlement of the pile head over time and determine the ultimate vertical compressive bearing capacity of the single pile. Vibrating wire stress gauges were installed at various cross-sections to collect force distribution data, and in situ static penetration tests were then performed to obtain the ultimate side friction resistance values of each soil layer, as presented in Table 3.

Table 3. Ultimate side friction resistance values of each soil layer (unit: kPa).

Stratum	Ultimate Side Friction Resistance	Static Penetration Tests
2-1	80	69.26
2-2	87	81.99
2-5	96.05	108.89
4-1	172.3	137.68

3. Study on Soil Displacement Effects During Pile Driving

This study used the extended Lade–Duncan yield criterion model, which effectively characterizes the stress–strain relationship of soft clay, to simulate the continuous deformation and compaction of clay layers during pile installation. The Coupled Eulerian–Lagrangian (CEL) method, which is suitable for large-deformation analysis, was used to model the undrained pile-driving process in soft clay, and the distribution patterns of soil displacement, horizontal stress, and pile penetration resistance were investigated in the surrounding soil during the pile-sinking process.

3.1. Cylindrical Cavity Expansion Problem

The cylindrical cavity expansion is commonly used to study pile driving process. The expansion under uniformly distributed internal pressure p is illustrated in Figure 9. When internal pressure p increases, the cylindrical zone surrounding the cavity transitions from an elastic state to a plastic state, and the plastic zone continues to expand with increasing internal pressure p . Let the initial radius of the cylindrical cavity be R_0 , the final expanded cavity radius after expansion be R_u , the maximum radius of the plastic zone be R_p , and the corresponding final internal pressure be p_u . The soil beyond R_p remains in an elastic equilibrium state. The cylindrical cavity expansion problem is a plane strain axisymmetric problem. The cylindrical cavity expansion theory, which was first proposed by Vesic [5] and developed by Chadwick [6] and Carter [7], has become a fundamental approach to addressing the impact of pile driving on the surrounding environment. This framework encompasses both cylindrical and spherical cavity expansion models. Classical theories assume that the soil behaves as an ideal elastoplastic material and is governed by the Tresca or Mohr–Coulomb yield criteria, which provide analytical solutions for cavity expansion under a uniform internal pressure p in an infinite soil mass based on elastoplastic theory. Subsequent studies have explored alternative constitutive models. For example, Bigoni and Laudiero [8] used a non-Mohr–Coulomb formulation for cylindrical cavity analyses, and Yu and Houlsby [9] highlighted the computational limitations in calculating clay volume strain. Further investigations by Yu [10] and Houlsby [11] examined the effects of pilot holes, which are a common field practice. In addition, Durban and Kubi [12] applied the Tresca model to analyze the problem, whereas Papanastasiou and Durban [13] used the Drucker–Prager model. Yu and Rowe [14] investigated a modified Mohr–Coulomb model, while Wang [15], Gong [16], and Jiang and Shen [17–19] studied hybrid Tresca and Mohr–Coulomb approaches. Therefore, this study focused on determining the side friction resistance in large-diameter prestressed high-strength concrete (PHC) pipe piles under pilot hole conditions.

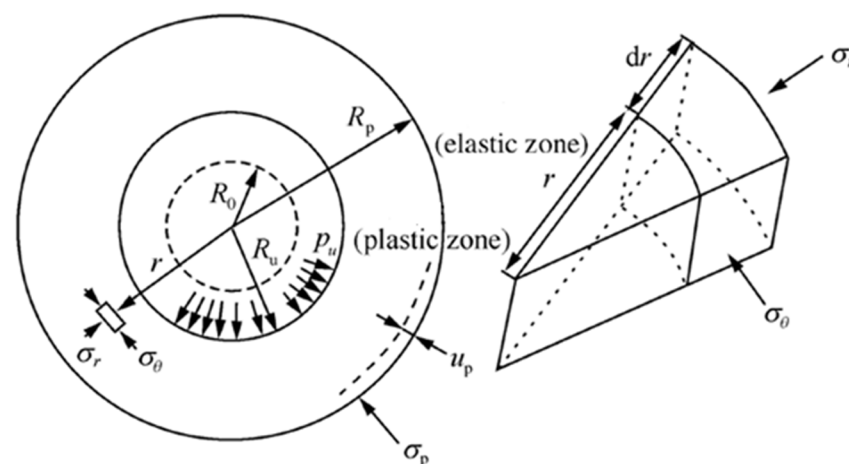


Figure 9. Expansion of the circular hole.

Based on the equilibrium differential equation, we have

$$\frac{d\sigma_r}{dr} + \frac{\sigma_r - \sigma_\theta}{r} = 0 \quad (1)$$

In Equation (1), σ_r is the radial stress, σ_θ is the normal stress, and r is the radius. From the geometric equation, the following holds:

$$\left. \begin{aligned} \varepsilon_r &= \frac{du_r}{dr} \\ \varepsilon_\theta &= \frac{u_r}{r} \end{aligned} \right\} \quad (2)$$

In Equation (2), ε_r is the radial strain, ε_θ is the normal stress, and u_r is the radial displacement.

The soil in the elastic zone conforms to the generalized Hook's law as follows:

$$\left. \begin{aligned} \varepsilon_r &= \frac{1-\mu^2}{E} \left(\sigma_r - \frac{1-\mu}{\mu} \sigma_\theta \right) \\ \varepsilon_\theta &= \frac{1-\mu^2}{E} \left(\sigma_\theta - \frac{1-\mu}{\mu} \sigma_r \right) \end{aligned} \right\} \quad (3)$$

In Equation (3), E is the elastic modulus, and μ is the Poisson ratio.

Based on the elastic theory, the elastic phase stress and displacement follow Equation (4):

$$\left. \begin{aligned} \sigma_r &= \frac{R_0^2 p}{r^2} \\ \sigma_\theta &= \frac{-R_0^2 p}{r^2} = -\sigma_r \end{aligned} \right\} \quad (4)$$

where p is the internal pressure generated in the cylinder, and R_0 is the initial hole radius of the cylinder.

The radial displacement is expressed as follows:

$$u = \frac{(1+\mu)}{E} r \sigma_r \quad (5)$$

3.2. Calculation of the Side Friction Resistance During Pile Driving Under Undrained Conditions Using the Extended Lade–Duncan Model

The Lade–Duncan model was first developed by Lade and Duncan based on true triaxial tests on sandy soils. This elastoplastic ideal constitutive model is suitable for stress–strain analyses of cohesionless soils. After revisions and refinements by Lade, this criterion has been widely applied in engineering practice. The yield criterion of the Lade–Duncan model differs from other strength criteria, and its form in terms of stress invariants is:

$$f = \frac{I_1^3}{I_3} = K_f \quad (6)$$

In Equation (6), K_f is the intensity parameter, and I_1 and I_3 are the first and third stress invariants of non-cohesive soil, respectively, which can be calculated using Equation (7):

$$\left\{ \begin{aligned} I_1 &= \sigma_1 + \sigma_2 + \sigma_3 \\ I_3 &= \sigma_1 \sigma_2 \sigma_3 \end{aligned} \right. \quad (7)$$

The original Lade–Duncan yield criterion is only applicable to cohesionless soils. For cohesive soils, a bonding stress σ_0 must be introduced for modification, which forms the extended Lade–Duncan criterion. The revised stress invariant form is as follows:

$$\frac{I_1^3}{I_3} = \frac{(3 - \sin \varphi_{tc})^3}{(1 + \sin \varphi_{tc})(1 - \sin \varphi_{tc})^2} \quad (8)$$

In Equation (8), I'_1 and I'_3 are the first and third stress invariants of cohesive soil, respectively:

$$\begin{cases} I_1 = (\bar{\sigma}_1 + \sigma_0) + (\bar{\sigma}_2 + \sigma_0) + (\bar{\sigma}_3 + \sigma_0) \\ I_3 = (\bar{\sigma}_1 + \sigma_0)(\bar{\sigma}_2 + \sigma_0)(\bar{\sigma}_3 + \sigma_0) \end{cases} \quad (9)$$

where $\bar{\sigma}_1$, $\bar{\sigma}_2$, and $\bar{\sigma}_3$ are the first, second, and third principal stresses of cohesive soil, respectively; σ_0 is the bonding stress, and $\sigma_0 = c \cot \varphi$; c is the cohesion; and φ is the internal friction angle of the material.

Under plane strain conditions, the clay damage criterion can be expressed as

$$\tau = c_{ps} + \bar{\sigma} \tan \varphi_{ps} \quad (10)$$

In Equation (10), c_{ps} is the cohesion force under planar strain conditions.

If the material is sand without a cohesive material, it is degraded to

$$\tau = \sigma \tan \varphi_{ps} \quad (11)$$

where σ_i and σ_0 are calculated as follows:

$$\sigma_i = \bar{\sigma}_i + \sigma_0 \quad (12)$$

$$\sigma_0 = c_{ps} \cot \varphi_{ps} \quad (13)$$

The assumed bonding stress under triaxial compression is calculated as follows:

$$\sigma_0 = c_{tc} \cot \varphi_{tc} \quad (14)$$

$$\sigma_i = \bar{\sigma}_i + c_{tc} \cot \varphi_{tc} \quad (15)$$

Therefore, the damage criterion of the cohesive soil under the plane strain condition based on the expanded Lade–Duncan criterion is as follows:

$$\bar{\sigma}_1 - \eta \bar{\sigma}_3 - (\eta - 1) \sigma_0 = 0 \quad (16)$$

where

$$\eta = \frac{1}{4} \left[\sqrt[3]{K} - 1 + \sqrt{\left(\sqrt[3]{K} - 1 \right)^2 - 4} \right]^2 \quad (17)$$

$$K = \frac{(3 - \sin \varphi_{tc})^3}{(1 + \sin \varphi_{tc})(1 - \sin \varphi_{tc})^2} \quad (18)$$

$$\sigma_0 = c_{tc} \cot \varphi_{tc} \quad (19)$$

3.3. Solution for the Extended Lade–Duncan Plastic Region Containing the Lead Hole

Substituting the extended Lade–Duncan model into Equation (1) yields

$$\frac{d\sigma_r}{dr} + \left(1 - \frac{1}{\eta}\right) \frac{\sigma_r}{r} + \left(1 - \frac{1}{\eta}\right) \frac{\sigma_0}{r} = 0 \quad (20)$$

Following Equation (20), one obtains

$$\sigma_r = c \frac{1}{r^\alpha} - \sigma_0 \quad (21)$$

where $\alpha = 1 - \frac{1}{\eta}$.

According to the boundary conditions $r = R_u$, $\sigma_r = p_u$, the stress solution for the available plastic zone is as follows:

$$\sigma_r = (p_u + \sigma_0) \left(\frac{R_u}{r} \right)^\alpha - \sigma_0 \quad (22)$$

$$\sigma_\theta = \frac{1}{\eta} (p_u + \sigma_0) \left(\frac{R_u}{r} \right)^\alpha - \sigma_0 \quad (23)$$

In low-permeability saturated clay, the pile-driving process can be considered an undrained compression process. Under undrained conditions, the soil is considered incompressible. If the changes in volume in the plastic material during the elastic phase and in the pile itself are ignored, the volume of the displaced soil during pile driving can be assumed to be equal to the sum of the volume change in the elastic zone and the soil plug volume.

$$\pi R_u^2 L - \pi R_0^2 L = \pi R_p^2 L - \pi (R_p - u_p)^2 L + \pi R_u^2 h \quad (24)$$

In Equation (24), L is the penetration depth of the PHC pipe pile, h is the length of the soil plug, R_u is the final borehole radius, R_p is the maximum radius of the plastic zone, R_0 is the initial borehole radius, u_p is the radial displacement at the outer boundary of the plastic zone, and R'_u is the radius of the soil plug after the borehole expansion is complete.

Based on Equation (24), one can ignore the higher order of u_p :

$$u_p = \frac{R_u^2 L - R_0^2 L - R_u^2 h}{2 R_p L} \quad (25)$$

From Equation (5) and the boundary conditions $r = R_p$, $\sigma_r = \sigma_p$, one obtains

$$u_p = \frac{(1 + \mu) R_p}{E} \sigma_p \quad (26)$$

From Equation (22), the radial stress at the elastic–plastic interface is

$$\sigma_p = (p_u + \sigma_0) \left(\frac{R_u}{R_p} \right)^\alpha - \sigma_0 \quad (27)$$

Substituting Equation (27) into Equation (26) yields

$$u_p = \frac{(1 + \mu) R_p}{E} \left[(p_u + \sigma_0) \left(\frac{R_u}{R_p} \right)^\alpha - \sigma_0 \right] \quad (28)$$

From Equation (4), at the elastic–plastic interface, we have

$$\sigma_r = -\sigma_\theta = \sigma_p \quad (29)$$

Substituting Equation (29) into Equation (28) yields

$$\sigma_r - \eta \sigma_\theta - (\eta - 1) \sigma_0 = 0 \quad (30)$$

Equations (29) and (30) are established together to obtain

$$\sigma_p = \frac{\eta - 1}{\eta + 1} \sigma_0 \quad (31)$$

Equations (25) and (26) are established together to obtain

$$\frac{R_u^2 L - R_0^2 L - R_u^2 h}{2R_p L} = \frac{(1 + \mu)R_p}{E} \sigma_p \quad (32)$$

Substituting Equation (31) into Equation (32) yields the plastic zone radius equation:

$$R_p = \sqrt{\frac{E(R_u^2 L - R_0^2 L - R_u^2 h)(\eta + 1)}{2(1 + \mu)(\eta - 1)\sigma_0 L}} \quad (33)$$

Combining Equations (27) and (31) yields

$$\frac{\eta - 1}{\eta + 1} \sigma_0 = (p_u + \sigma_0) \left(\frac{R_u}{R_p} \right)^\alpha - \sigma_0 \quad (34)$$

The limit reaming pressure p_u is calculated as follows:

$$p_u = \sigma_0 \left[\frac{2\eta}{\eta + 1} \left(\frac{R_p}{R_u} \right)^\alpha - 1 \right] \quad (35)$$

Substituting Equation (33) into Equation (35) yields the limit hole expansion pressure of pile sinking p_u :

$$p_u = \sigma_0 \left[\frac{2\eta}{\eta + 1} \left(\sqrt{\frac{E(R_u^2 L - R_0^2 L - R_u^2 h)(\eta + 1)}{2(1 + \mu)(\eta - 1)\sigma_0 L R_u^2}} \right)^\alpha - 1 \right] \quad (36)$$

In Equation (36), E is the elastic modulus of cohesive soil, R_u is the external radius of the driven pile, R_0 is the internal pilot hole radius, R'_u is the radius of the soil plug inside the pile, L is the driving depth of the PHC tubular pile, and h is the length of the soil plug. For the other parameters, refer to Equation (37):

$$\left\{ \begin{array}{l} \sigma_0 = c_{tc} \cot \varphi_{tc} \\ \eta = \frac{1}{4} \left[\sqrt[3]{K} - 1 + \sqrt{\left(\sqrt[3]{K} - 1 \right)^2 - 4} \right]^2 \\ K = \frac{(3 - \sin \varphi_{tc})^3}{(1 + \sin \varphi_{tc})(1 - \sin \varphi_{tc})^2} \\ \alpha = 1 - \frac{1}{\eta} \end{array} \right. \quad (37)$$

where c_{tc} and φ_{tc} are the cohesion and internal friction angle, respectively, under triaxial compression conditions, the values of which can be obtained through triaxial compression tests. Similarly, the radial displacement radius of the lateral side can be obtained by combining Equations (25) and (33):

$$u_p = \sqrt{\frac{\sigma_0(1 + \mu)(R_u^2 L - R_0^2 L - R_u^2 h)(\eta - 1)}{2EL(1 + \eta)}} \quad (38)$$

4. CEL Finite Element Model for Assessing Soil Squeezing Effect During Pile Driving

4.1. Introduction to the CEL Method

The Coupled Eulerian–Lagrangian (CEL) method integrates the advantages of the Lagrangian and Eulerian approaches to address large-deformation problems in computational grids. This method enables interactions between Eulerian elements (for parts that experience large deformations) and Lagrangian elements (for relatively rigid parts) in the

same model. During the CEL analysis, the Eulerian Volume Fraction (EVF) describes the “trace” of Eulerian materials. The Eulerian time increment algorithm uses the Lagrange-plus-remap algorithm [20]: first, in the Lagrangian phase of the time increment, nodes are temporarily fixed to the material, and elements deform with the material; then, in the Eulerian phase, deformed elements are re-meshed (restored to their original state) to compute the material flow through them.

4.2. Finite Element Model

The pile installation process is typically characterized as a plane strain problem. To mitigate computational demands, a quarter-cylinder model was employed in the analysis, as illustrated in Figures 10 and 11. Boundary constraints impose restrictions on deformation along the XZ and YZ planes. This approach is applicable only when the soil is significantly weaker than the pile, a condition that is generally satisfied during the installation of prestressed high-strength concrete (PHC) piles.

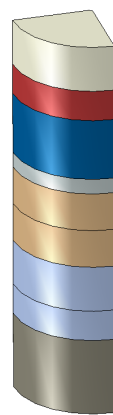


Figure 10. Stratified materials in the Eulerian domain (empty layer at the top).

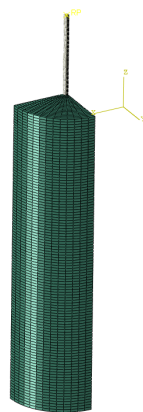


Figure 11. Global mesh.

The pipe pile had an outer diameter (denoted as D hereafter) of 1.2 m and an inner diameter of 0.9 m. Prototype pile No. 1, with a length of 20 m, was selected for simulation. The soil domain had a thickness of 50 m and was filled with the corresponding materials ($EVF = 1$) based on geological data. A 5 m thick material-free zone ($EVF = 0$) was reserved above the soil to enable upward material flow during pile penetration (stratification details are presented in Figure 9), and the soil radius was 12 m ($10D$). The mesh was refined around the pile (Figure 12), where the minimum element size was 0.075 m ($\frac{1}{16}D$), and the model contained 54,900 elements and 63,094 nodes. The soil elements were Eulerian (type EC3D8R), and the pile was simulated as a discrete Lagrangian rigid body (type R3D4).

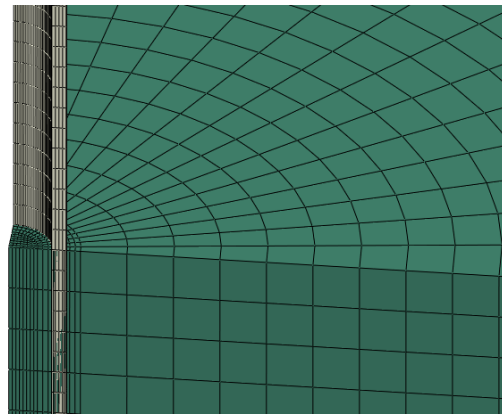


Figure 12. Local mesh refinement. Green meshes denote the Eulerian domain, while white meshes represent the Lagrangian domain.

The Coupled Eulerian-Lagrangian (CEL) method is implemented using the ABAQUS/Explicit solver, thereby circumventing convergence issues typically associated with implicit analysis. The accuracy of the CEL approach is highly influenced by mesh size and loading velocity. Qing et al. investigated the effect of mesh size in the context of soil cutting processes [21]. They recommended that the maximum size of the Lagrangian mesh should be at least one-fourth of the characteristic length of the Lagrangian domain, while the Eulerian mesh should be refined to half the size of the Lagrangian mesh. In the present simulation, the maximum Lagrangian mesh size is set to 0.2 m, which corresponds to one-fourth of the pile arc length (0.94 m). The Eulerian mesh size is approximately 0.075 m, representing a refinement of approximately 2.7 times relative to the Lagrangian mesh (0.2 m).

The pile-driving velocity significantly influences both the accuracy and computational efficiency of CEL simulations. Lower velocities generally improve accuracy but at the expense of increased computational time, necessitating a balanced selection. Tang [22] examined appropriate pile installation velocities and recommended a range between 0.3 m/s and 0.7 m/s. In this simulation, a velocity of 0.5 m/s is adopted to realistically represent the actual pile installation process.

Boundary conditions:

1. Soil base: Z-direction velocity constrained.
2. Cylindrical surface: X- and Y-direction velocity constrained.
3. Symmetry planes: Normal-direction velocity constrained.

Analysis procedure:

1. Apply initial geostatic stress to the soil while constraining all pile degrees of freedom.
2. Release the Z-direction freedom of the pile to enable the continuous downward penetration at a constant velocity.

4.3. Computational Parameters

The soil material parameters (see Table 4) were derived from geotechnical reports, and the Mohr–Coulomb constitutive model was adopted. The pile–soil interaction was modeled using the general contact algorithm in Abaqus, which automatically tracks interfaces. The Coulomb friction model governs the tangential behavior (hard contact in the normal direction to prevent penetration), and the friction coefficient is defined as

$$\mu = K_0 \tan \delta$$

where

K_0 is the coefficient of earth pressure at rest ($K_0 = 1 - \sin \phi'$);

ϕ' is the effective internal friction angle;

δ is the interface friction angle, and $\delta = \left(\frac{1}{3} \sim \frac{2}{3}\right)\phi'$.

Table 4. Physical and mechanical parameters of each stratum.

Stratum	Density (g/cm ³)	Elastic Modulus (MPa)	Poisson Ratio	Cohesion Force (kPa)	Internal Friction Angle
2-1	1.84	24	0.27	24.3	10.5
2-2	1.93	22	0.25	12.4	11.5
2-5	1.94	40	0.3	13	22
2-4	1.98	21	0.21	25.2	25
2-72	1.98	17	0.32	-	25
4-1	1.97	42	0.32	27.5	26

5. Analysis of the Calculation Results

5.1. CEL Model Calculation Results

The calculation results show that when the pile penetrated the soil, the stratum material moved downward due to frictional forces and intruded into the lower stratum. Simultaneously, the surrounding soil was compressed and heaved upward. For example, in stratum 2-1, the stratigraphic intrusion caused by friction was 1 m, which indicates extremely high friction between the pile and soil (Figure 13). Figures 14–18 show the extracted X- and Z-directional stresses in each soil layer. The horizontal pressure S11 and vertical frictional force S33 between the pile and soil clearly exhibit uneven distributions under different strata.

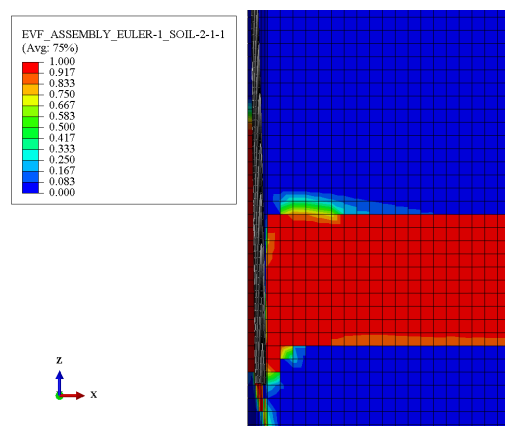


Figure 13. Stratum 2-1 material filling situation.

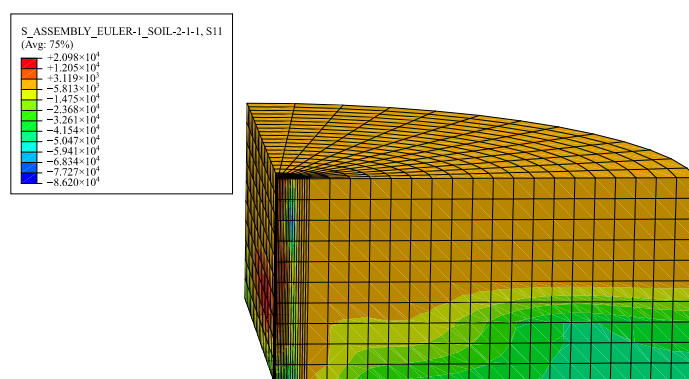


Figure 14. Stress distribution in stratum 2-1 in the X direction.

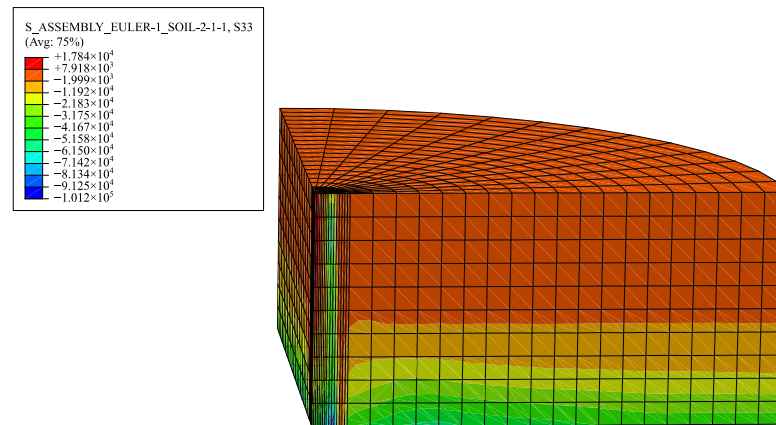


Figure 15. Stress distribution of stratum 2-1 in the Z direction.

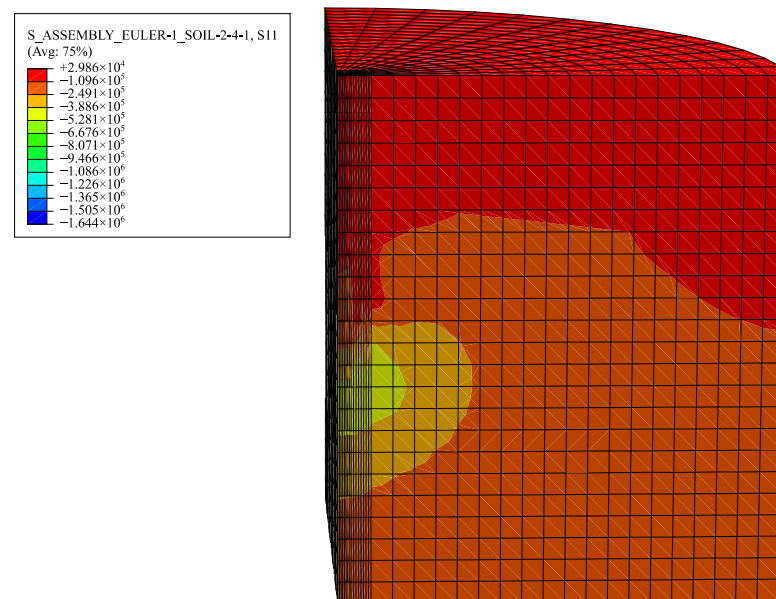


Figure 16. Stress distribution in stratum 2-4 in the X direction.

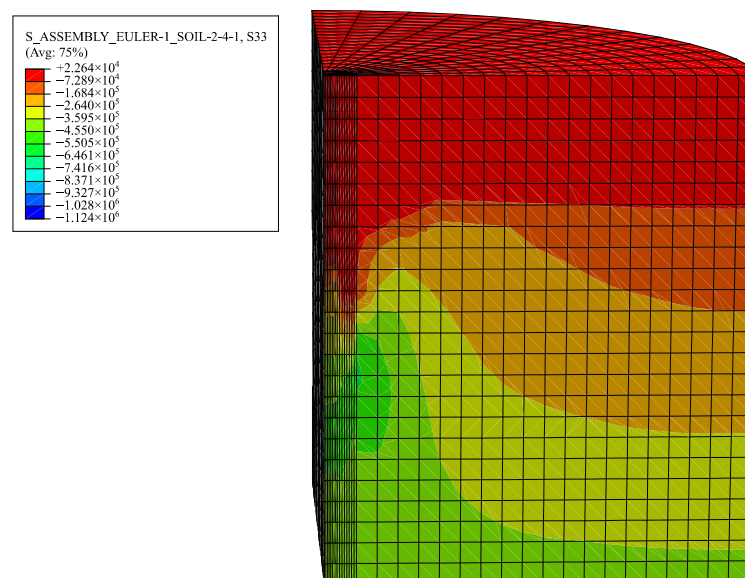


Figure 17. Stress distribution of stratum 2-4 in the Z direction.

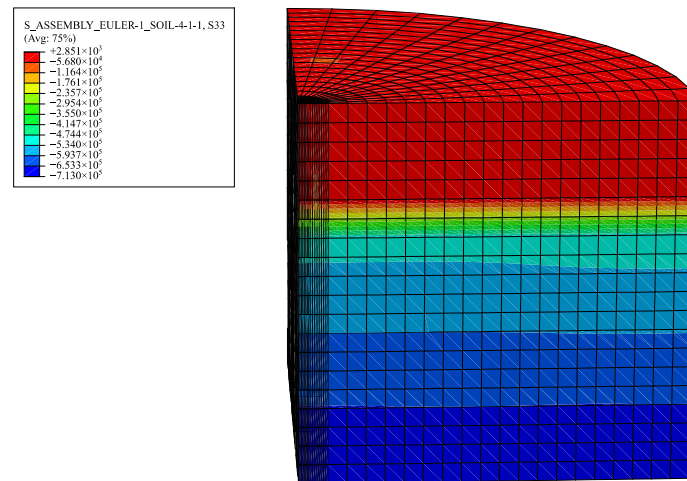


Figure 18. Stress distribution of stratum 4-1 in the Z direction.

5.2. Ultimate Frictional Resistance of Soil Surrounding the Pile

By substituting material parameters into Equation (36), we calculated the theoretical solutions for the ultimate cavity expansion pressure and ultimate frictional resistance in each clay layer. Table 5 shows the S33 stress components (X–Z plane) of the CEL model for each soil layer, and Table 6 shows the computed relative errors between the theoretical solutions and the CEL model in comparison to the static penetration test results. The CEL calculations had an average error of 22% and a maximum relative error of 50%, whereas the theoretical model had an average error of 5% and a maximum relative error of 22%. Thus, the extended Lade–Duncan model under undrained conditions achieves higher accuracy than the Mohr–Coulomb model. However, the CEL model improved the precision in calculating the lateral frictional resistance in deeper soil layers, which led to more accurate stress predictions at the pile tip.

Table 5. Ultimate side friction resistance of each stratum (unit: kPa).

Stratum	Theoretical Solution for the Limit Reaming Pressure	Theoretical Solution for the Ultimate Friction Resistance	CEL Model Results	Static Penetration Test
2-1	742.75	72.70	100.2	69.26
2-2	385.90	50.08	61.4	41
2-5	480.63	76.62	86.2	81.99
2-4	504.50	84.90	112.4	108.89
4-1	706.95	120.63	171.3	156.77

Table 6. List of relative errors compared with static penetration test data.

Stratum	Theoretical Solution	CEL Model Results	Static Penetration Test
2-1	105%	145%	100%
2-2	122%	150%	100%
2-5	93%	105%	100%
2-4	78%	103%	100%
4-1	77%	109%	100%

6. Discussion

Published experimental or numerical studies investigating large-diameter (≥ 1.2 m) PHC pipe piles in high-speed railway foundations remain limited. The observed relative er-

ror of approximately 50% in surface strata from CEL simulations using the Mohr–Coulomb model underscores the need for improved modeling approaches. The findings presented in this manuscript contribute to a more accurate understanding of the soil displacement effects induced by PHC pipe pile installation.

In engineering applications of large-diameter PHC pipe piles, the accurate prediction of pile–soil interaction is often hindered by the inability of traditional methods to handle large deformations and complex constitutive behaviors. Previous studies have predominantly relied on the Mohr–Coulomb criterion, which often fails to precisely capture the non-linear stress–strain response of soft clays under high-stress undrained conditions. By employing the extended Lade–Duncan yield criterion and the CEL method, this study reveals that the soil displacement effect is a governing factor that enhances lateral frictional resistance, particularly in deep soil layers. This micromechanical understanding enables a more rational interpretation of force transmission compared to empirical approaches based solely on simple interface friction.

From an engineering standpoint, these results underscore the practical value of the established “Field Test–Theoretical Analysis–CEL Simulation” framework. The comparison between field static penetration tests and numerical results demonstrates that the CEL method effectively overcomes mesh distortion issues common in traditional FEM, providing a robust tool for analyzing penetration resistance in multi-layered strata.

However, several limitations should be acknowledged. The analysis presented in this manuscript is based on in situ tests and numerical simulations. The physical and mechanical parameters of the strata were obtained from laboratory tests and geological surveys. The simulation configurations were designed according to typical on-site test piles. The vertical and lateral variability of the strata should be taken into account when applying the simulation results. The CEL simulations exhibited a relative error of approximately 50% in the surface layer compared to theoretical solutions. This discrepancy suggests that in the numerical model, upper stratum materials flowed unconfined away from the original mesh, whereas in the actual field tests, surface loads from construction machinery (e.g., vehicles and dozers) likely constrained this overflow. Therefore, while the model excels in deep-depth analysis, the boundary conditions for surface layers require careful refinement in future modeling to avoid overestimation of material flow. The CEL model has higher accuracy in deep stratum than theoretical solutions. The combination of both CEL and theoretical solutions would be beneficial in understanding the soil displacement effect in large-diameter PHC pipe piles.

To enhance practical transferability, future studies would benefit from further validating this framework against broader geological scenarios and integrating time-dependent consolidation effects. Nevertheless, the current findings confirm that a holistic approach combining advanced constitutive models with large-deformation numerical techniques is essential for the rational design and construction control of large-diameter pile foundations.

The CEL data and static penetration test show that surface loads such as vehicles and dozers may reduce the overflow of stratum material. The site test produced results consistent with the theoretical solution. The CEL model should be carefully examined for overflow in such situations.

7. Conclusions

The pile-driving process constitutes a complex mechanical phenomenon. Using the extended Lade–Duncan model under undrained conditions and analyzing the post-driving frictional resistance of large-diameter PHC pipe piles, reverse analysis revealed that the soil displacement effect critically enhanced pile–soil interfacial friction. This approach also enables relatively precise calculations of the lateral frictional resistance across varying

strata. Through cone penetration tests, theoretical analysis, and numerical simulations, we obtained the following conclusions:

1. Enhanced Model Accuracy: For pile driving in undrained clay strata, the extended Lade–Duncan model is more precise than the Mohr–Coulomb model, reducing the relative error from 22% (Mohr–Coulomb) to 5%.
2. Advantages of the CEL Method: The Coupled Eulerian–Lagrangian (CEL) method effectively replicates the lateral frictional resistance in deep soil layers during pile-driving simulations and considerably improves the analytic accuracy at the pile tip.
3. Requirement for a Holistic Approach: To accurately capture soil displacement effects in complex geological strata during large-diameter PHC pile installation, the combination of theoretical calculations and CEL-based numerical modeling remains essential for comprehensive analysis. The analytical results indicate that the clay stratum below a depth of 10 m demonstrates enhanced suitability for numerical simulations utilizing the CEL approach.

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Abbreviation

The following abbreviation is used in this manuscript:

PHC Prestressed high-intensity concrete

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