

Article

Theoretical Analysis of Drilling Unloading and Pile-Side Soil Pressure Recovery of Nonsqueezing Pipe Piles Installed in K_0 -Consolidated Soils

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Abstract: Drilling with prestressed concrete (DPC) pipe pile is a nonsqueezing pile sinking technology, employing drilling, simultaneous pile sinking, a pipe pile protection wall, and pile side grouting. The unloading induced by drilling, the pipe pile supporting effect, and the dissipation of the negative excess pore-water pressure after pile sinking, all of which have significant effects on the recovery of soil pressure on the pile side, are the main concerns of this study, which aim to establish a method to reasonably evaluate the timing selection of pile side grouting. The theoretical solutions for characterizing the unloading and dissipation of the negative excess pore-water pressure are presented based on the cylindrical cavity contraction model and the separated variable method. By inverse-analyzing the measured initial pore pressure change data from borehole unloading, initial soil pressures on the pile side of each soil layer are determined using the presented theoretical solutions. Then, the presented theoretical solutions were verified through a comparative analysis with the corresponding measured results. Moreover, by introducing time-dependent coefficients α_{t1} and α_{t2} to characterize the pore pressure dissipation and rheology effects, the effects of the negative excess pore-water pressure dissipation on the pile-side soil pressure recovery are discussed in detail.

Keywords: DPC pipe pile; drilling unloading; soil pressure recovery; cylindrical cavity contraction; negative excess pore water pressure



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1. Introduction

Hammering (or hydrostatic pressing) PHC pipe piles during construction have significant squeezing effects, which tend to cause destructive impacts on the surrounding structures and underground pipelines [1–4]. DPC pipe pile is a nonsqueezing pile sinking technology that employs drilling, simultaneous pile sinking, and a pipe pile protection wall [5,6]. The DPC pipe pile technology effectively solves the technical problem of large diameter (800–1400 mm) PHC pipe piles that are difficult to sink due to the squeezing effect of hammering technology (or hydrostatic pressing technology). The paddle-reaming bit used with the pile sinking equipment (Figure 1) can drill into moderately and slightly weathered rock to form a hole with a diameter of approximately 20 mm larger than that of pipe piles, thereby achieving a pipe pile socketing rock without squeezing effect [7–9]. Compared with the hammering (or hydrostatic pressing) of PHC pipe piles, the soil disturbance included by the sinking of DPC pipe piles is reduced by 78% to 81% [9]. However,

for vertical load bearing piles, it is required to seal the pile end and grout the pile side of DPC pipe piles after pile sinking. The pile-sinking construction induces stress release of the soil around piles, resulting in an unloading effect that not only impacts the adjacent building structure but also influences the subsequent grouting reinforcement on the pile side. The unloading and the soil pressure recovery during the interval between sinking and grouting are crucial pieces of information required for pile-side grouting.



Figure 1. Construction equipment of DPC pipe piles: (a) pile driving equipment, (b) paddle-reaming bit 1, and (c) paddle-reaming bit 2.

Westergard [10], Gholami et al. [11], Li et al. [12], Abbasi et al. [13], and Zhao et al. [14] presented elastic-plastic solutions for unloading deformations by considering the soil around piles as a semi-infinite body and analyzed the stability problem of the whole wall of a slurry drilled pile construction based on different strength criteria. However, it is noteworthy that the unloading deformation of DPC pipe piles is significantly affected by the pipe pile protection wall and differs from that of slurry drilled piles. Furthermore, previous studies have pointed out that the elastic rebound caused by transient unloading induces a reduction in pore water pressure, thereby generating negative excess pore water pressure (NEPWP). For example, Hao et al. [15] analyzed the rebound mechanism of a foundation after overload unloading and the process of NEPWP generation and dissipation. Lee and Ng [16] and L'Heureux et al. [17] employed a numerical model to investigate the generation and evolution law of NEPWP in the region affected by soil excavation. Shi et al. [18] and Zhang et al. [19] investigated the rebound-absorption deformation characteristics of soft clay specimens, and they discussed the changes to the consolidation and rebound coefficients. Moreover, Shi et al. [20] and Liang et al. [21] presented theoretical solutions for NEPWP dissipation of a soft clay foundation under linear unloading. These studies have demonstrated that the principle of NEPWP dissipation is similar to that of excess pore water pressure (EPWP) dissipation and is the inverse process of EPWP dissipation.

Zhao et al. [22] derived the series solution for EPWP dissipation in the soil around the pile at any position and any moment based on cylindrical cavity expansion theoretical solutions. Li et al. [23] gave EPWP and its evolution basic solutions based on the modified Cambridge model improved by the SMP criterion. Li et al. [24] discussed the effect of EPWP dissipation in the soil around the pile on the soil pressure. Their studies primarily focus on EPWP dissipation and its impact on soil pressure exerted on the pile side, disregarding the soil rheology effect. However, the pile-side soil pressure recovery of DPC pipe piles, occurring under unloading and pipe pile protection wall conditions, is not

only dependent on NEPWP dissipation but also on soil rheology [25–27]. Therefore, the mechanical mechanism governing soil pressure changes of DPC pipe piles is different from that of Hammering PHC pipe piles, necessitating further study.

This paper focuses on the unloading and pile-side soil pressure recovery of DPC pipe piles, disregarding the pile spacing effect. First, theoretical solutions for mechanical state change caused by drilling unloading of DPC pipe piles are presented based on the cylindrical cavity contraction model. Second, the series solutions for NEPWP dissipation are derived based on the separated variable method. Then, the initial soil pressure of the pile side is determined via inversion analysis of the measured initial pore pressure change data that applied the presented theoretical solutions, and the rationality of the theoretical solution is verified. Finally, the effect of NEPWP dissipation on the soil pressure on the pile side is discussed by introducing the time-dependent coefficient.

2. Mechanical Mechanism and Basic Assumptions

2.1. Mechanical Mechanism

The mechanical state changes of soil around the pile caused by pile sinking primarily involve two stages (Figure 2): the instantaneous unloading deformation stage and the NEPWP dissipation and soil pressure recovery on the pile side stage.

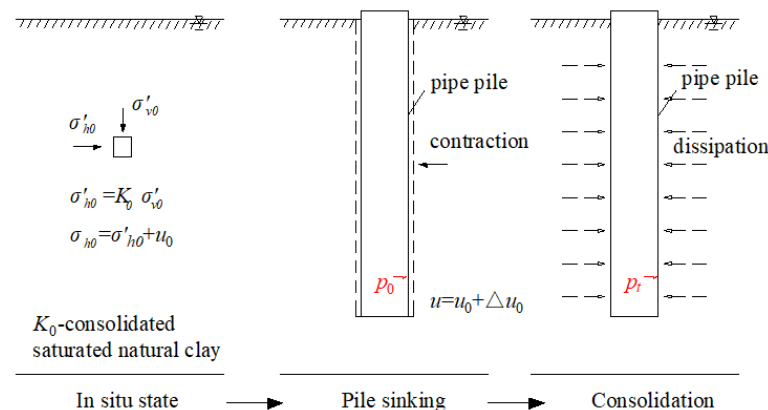


Figure 2. Schematic diagram of mechanical state change in soil around piles.

As shown in Figure 2, the in situ state of saturated clayey soil is in K_0 consolidation with an in situ static pore water pressure of u_0 , in which the initial horizontal effective stress σ'_{h0} is equal to the product of the coefficient of earth pressure at rest and the initial vertical effective stress, $K_0\sigma'_{v0}$, and the initial horizontal total stress, $\sigma_{h0} = K_0\sigma'_{v0} + u_0$. The drilled hole diameter of the DPC pipe pile is slightly larger than the pile diameter; thus, the pile sinking construction causes stress release and accompanying contraction deformation in the soil around the pile, thereby disturbing the soil around the pile. Under the unloading condition, the borehole wall pressure is unloaded from σ_{h0} to the initial soil pressure on the pile's side p_0 . When p_0 is small, the pile-side pressure is insufficient to support the unloading deformation; thus, plastic deformation occurs, leading to the generation of NEPWP with the plastic region.

After pile sinking, the soil pressure on the pile side increases gradually with the soil consolidation and rheology, exhibiting time-dependent characteristics.

Compared with the unloading deformation observed in slurry drilled pile construction [11–14,17] and the squeezing deformation encountered during hammering (or hydrostatic pressing) of PHC pipe piles construction [22,23], the boundary conditions for mechanical state changes caused by DPC pipe pile construction differ. The pile hole, which is 10 mm larger than the outer radius diameter of the pipe pile, is formed by drilling a paddle-reaming bit below the pipe pile. The pipe piles play a functional role in the protection of the borehole wall. Most of the rock and soil slags are discharged from the inner cavity of the pipe piles, and a small amount of slag under the rotating action of the

paddle-reaming bit enters the gap between the piles and the soil body. These slugs in the gap also play a role in protecting the borehole wall. As a result, the initial soil pressure on the pile side cannot be determined directly. Thus, the unloading deformation induced by the pile sinking construction has an undetermined force boundary condition, which differs from that determined for slurry drilled pile construction. In addition, part of the unloaded deformed soil will be excavated twice by the reaming bit, thereby generating a secondary contraction deformation. Here, it is impossible to determine the displacement boundary condition directly, which differs from that determined for hammering (or hydrostatic pressing) during PHC pipe pile construction. From theories of cylindrical cavity expansion [22,23] to cylindrical cavity contraction [11–14,17], at least one of the force boundary conditions and the displacement boundary condition must be known for the solution of the mechanical state changes in the soil around the pile.

2.2. Basic Assumptions

In this paper, the soil around the pile is considered to be semi-infinite. The unloading deformation is simplified as a cylindrical cavity contraction problem, and the following basic assumptions are considered:

1. The homogeneous soil is in a K_0 consolidated state is affected by the initial horizontal and vertical effective stresses σ'_{h0} and σ'_{v0} , respectively, and is saturated with an initial pore water pressure u_0 . The initial horizontal total stress is given as $\sigma_{h0} = K_0 \sigma'_{v0} + u_0$;
2. The unloading deformation of the soil around the piles is carried out under plane strain and undrained conditions, and the soil particles and pore water are incompressible;
3. The initial soil pressure on the pile side after the pile sinking is known to be p_0 ;
4. The soil obeys Hooke's law, and the small strain assumption in the elastic state [21,24] and the yield and elastoplastic (E–P) characteristics of the soil are modeled by the Mohr–Coulomb model.

3. Unloading Deformation Induced by Pile Sinking

The unloading deformation induced by the sinking of DPC pipe piles is shown in Figure 3, including vertical and plan views. This phenomenon is simplified as an undrained cylindrical cavity contraction problem, which has been analyzed using the total stress method. Under the unloading condition of the borehole size r_0 , the borehole wall pressure is reduced from σ_{h0} to p_0 . In a cylindrical coordinate system, the stress components of a typical soil element are radial stress σ_r and tangential stress σ_θ . Here, if p_0 is small, the soil around the pile may deform plastically, and then it is divided into plastic and elastic regions, where the plastic region radius is denoted r_y .

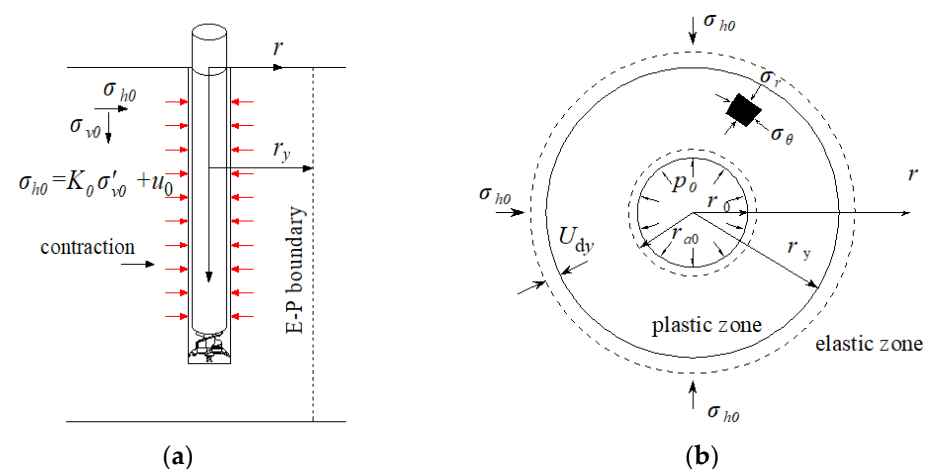


Figure 3. Contraction problem induced by pile sinking of DPC pipe pile in saturated K_0 consolidated soil: (a) vertical view and (b) plan view.

The stress components in the entire region satisfy the following equilibrium equation:

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

where r is the distance from a point to the center axis.

In the elastic region ($r > r_y$), the soil stress–strain relationship obeys Hooke’s law; thus, the theoretical solutions for the radial displacement and radial stress fields are obtained using the elastic physical and geometrical relationships as follows [9]:

$$U_r = \frac{1}{2G} [(\sigma_{ry} - \sigma_{h0})] \left(\frac{r_y}{r} \right)^2 r; \quad (2a)$$

$$\sigma_r = \sigma_{h0} + (\sigma_{ry} - \sigma_{h0}) \left(\frac{r_y}{r} \right)^2; \quad (2b)$$

$$\sigma_\theta = \sigma_{h0} - (\sigma_{ry} - \sigma_{h0}) \left(\frac{r_y}{r} \right)^2. \quad (2c)$$

Here, σ_{ry} , U_r , and G are the radial stress at the elastic–plastic (E–P) boundary, the radial displacement, and the shear modulus, respectively.

At the E–P boundary, by integrating the Mohr–Coulomb model,

$$\sigma_r = \sigma_\theta \frac{1 - \sin \varphi}{1 + \sin \varphi} - \frac{2c \cos \varphi}{1 + \sin \varphi}, \quad (3)$$

and $\sigma_r + \sigma_\theta = 2\sigma_{h0}$, the radial stress is obtained as follows:

$$\sigma_{ry} = (1 - \sin \varphi) \sigma_{h0} - c \cos \varphi. \quad (4)$$

Here,

$$\sigma_{h0} = K_0 \sigma'_{v0} + u_0 = (1 - \sin \varphi') \gamma' z + u_0, \quad (5)$$

where c , φ are the cohesion force and friction angle, and γ' and z are the effective unit weight and buried depth, respectively.

In the plastic region ($r_0 < r \leq r_y$), according to the force equilibrium and displacement coordination conditions, the theoretical solutions for the radial displacement and radial stress fields are obtained using Equations (1) and (3) as follows:

$$U_r = -\frac{1}{2G} (c \cos \varphi + \sigma_{h0} \sin \varphi) \left(\frac{r_y}{r} \right)^2 r; \quad (6a)$$

$$\sigma_r = (\sigma_{ry} + c \cot \varphi) \left(\frac{r}{r_y} \right)^{\frac{2 \sin \varphi}{1 - \sin \varphi}} - c \cot \varphi; \quad (6b)$$

$$\sigma_\theta = \frac{1 + \sin \varphi}{1 - \sin \varphi} (\sigma_{ry} + c \cot \varphi) \left(\frac{r}{r_y} \right)^{\frac{2 \sin \varphi}{1 - \sin \varphi}} - c \cot \varphi. \quad (6c)$$

Here, r_y is obtained as follows:

$$r_y = \left(\frac{p_0 + c \cot \varphi}{\sigma_{ry} + c \cot \varphi} \right)^{\frac{1 - \sin \varphi}{2 \sin \varphi}} r_0. \quad (7)$$

Note that the soil around the pile has not deformed plastically if $r_y \leq r_0$. Thus, the calculation of the displacement and stress field are performed using Equation (2), where σ_{ry} and r_y are replaced by p_0 and r_0 , respectively.

From Equation (2), we obtain $p = 0.5(\sigma_r + \sigma_\theta) = \sigma_{h0}$, which indicates that the average principal stress in the elastic region is constant, and there is no NEPWP. From Equation (6), we obtain $p = 0.5(\sigma_r + \sigma_\theta) < \sigma_{h0}$, which indicates that the average principal stress in the plastic region is reduced. According to the effective stress principle, the effective stress

under instantaneous unloading conditions does not change. The reduction of average principal stress is primarily manifested as that of the pore water pressure, thereby generating NEPWP. Here, using Equations (6b) and (6c), NEPWP is expressed as follows:

$$\Delta u_0 = \sigma_{h0} + c \cot \varphi - (\sigma_{h0} + c \cot \varphi) \left(\frac{r}{r_y} \right)^{\frac{2 \sin \varphi}{1 - \sin \varphi}}, \quad (8)$$

where Δu_0 is the initial NEPWP. Note that Δu_0 is a function of r along the radial direction with $r^{-2 \sin \varphi / (1 + \sin \varphi)}$ decay from the borehole wall to the E–P boundary.

4. Pile-Side Soil Pressure Recovery Induced by NEPWP Dissipation

4.1. NEPWP Dissipation

According to relevant literature [15,18–21], NEPWP dissipation is equivalent to the reverse process of EPWP dissipation. Here, the extent of the plastic region is small, relative to the longitudinal depth of the pile; thus, it is postulated that NEPWP dissipation occurs primarily in the radial direction. The governing equation for NEPWP dissipation is expressed as follows [22,28]:

$$\frac{\partial u}{\partial t} = C_{ve} \frac{1}{r} \frac{\partial}{\partial r} \left(r \frac{\partial u}{\partial r} \right), \quad (9)$$

where

$$C_{ve} = \frac{k_v(1+e)}{a_v r_w}. \quad (10)$$

Here, C_{ve} , k_v , and a_v are the consolidation coefficient, permeability coefficient, and expansion coefficient, respectively; r_w , e are the water unit weight and pore ratio, respectively.

The initial condition for solving Equation (9) should be the NEPWP immediately after pile sinking, which is given by Equation (8). Assuming the pile is impermeable, the boundary conditions used to solve Equation (9) can be given as follows [21,24]:

$$\left. \frac{\partial u}{\partial r} \right|_{r=r_0} = 0 (t > 0); \quad (11)$$

$$u|_{r=r_y} = 0 (t > 0). \quad (12)$$

The general solution of the governing equation, Equation (9), using the variable separation method was presented by Liang et al. as follows [21]:

$$\Delta u(r, t) = \sum_{n=1}^{\infty} A_n M_0 e^{-\alpha_n^2 C_{ve} t}, \quad (13)$$

where $\Delta u(r, t)$ is NEPWP at any time and any radial coordinates after consolidation. Here, the undetermined coefficients M_0 , A_n , and α_n in Equation (9) can be determined as follows [21]:

$$M_0 = J_0(\alpha_n r) - \frac{J_1(\alpha_n r_0)}{Y_1(\alpha_n r_0)} Y_0(\alpha_n r), \quad (14)$$

$$A_n = \frac{\int_{r_0}^{r_y} u_0(r) M_0(a_n r) r dr}{\int_{r_0}^{r_y} M_0^2(a_n r) r dr}, \quad (15)$$

$$J_0(\alpha_n r_y) Y_1(\alpha_n r_0) = J_1(\alpha_n r_0) Y_0(\alpha_n r_y), \quad (16)$$

where J_0 and J_1 are the first kind Bessel functions of zero order and first order, and Y_0 and Y_1 are the second kind Bessel functions of zero order and first order, respectively.

4.2. Pile-Side Soil Pressure Recovery

After pile sinking, the soil pressure on the pile side gradually increases from the initial soil pressure on the pipe wall, p_0 , due to NEPWP dissipation and soil rheology. Over a

sufficient duration, the soil pressure on the pile side reaches the initial horizontal total stress, σ_{h0} . Introducing a time-dependent coefficient, the pile-side soil pressure is expressed as follows:

$$p_t = \alpha_t \sigma_{h0}, \quad (17)$$

where α_t is the time-dependent coefficient. Here, if $t = 0$ and $p_t = p_0$, then the lower bound of α_t is obtained as $\alpha_{t=0} = p_0/\sigma_{h0}$. If $t = \infty$ and $p_t = \sigma_{h0}$, then the upper bound of α_t is obtained as $\alpha_{t=\infty} = 1$. Note that the range of α_t is $p_0/\sigma_{h0} \sim 1$.

By separating NEPWP dissipation and soil rheology effects, the time-dependent coefficient can be expressed as follows:

$$\alpha_t = \alpha_{t1} + \alpha_{t2}, \quad (18)$$

where α_{t1} , α_{t2} are NEPWP dissipation and rheology effect coefficients, respectively. If the soil rheological effect is not taken into account, the pile-side soil pressure at $t = \infty$ is $p_0 + u(r_0, 0)$, then the upper bound of α_{t1} is $[p_0 + u(r_0, 0)]/\sigma_{h0} < 1$. Accordingly, $\alpha_{t1} = p_0/\sigma_{h0} \sim [p_0 + u(r_0, 0)]/\sigma_{h0}$ and $\alpha_{t2} = [p_0 + u(r_0, 0)]/\sigma_{h0} \sim 1$.

Using Equations (8), (17), and (18), the coefficient α_{t1} is expressed as follows:

$$\alpha_{t1} = \frac{1}{\sigma_{h0}} [p_0 + \Delta u_0 \Big|_{r=r_0} - \Delta u(t) \Big|_{r=r_0}], \quad (19)$$

where $\Delta u_0 \Big|_{r=r_0}$ is the initial NEPWP at $r = r_0$ and $\Delta u(t) \Big|_{r=r_0}$ is the NEPWP at any time of $r = r_0$.

5. Determination of Initial Pile-Side Soil Pressure and Model Validation

The initial pile-side soil pressure is required to calculate the unloading deformation using Equations (2) and (6); however, the initial pile-side soil pressure is not obtained directly due to pipe pile protection wall effects. In the following, the initial pile-side soil pressure in each soil layer is determined first by inverse analyzing the measured initial pore pressure data from a field test. Then, NEPWP in the soil around piles at different moments is calculated using theoretical solutions, which are subsequently compared with the corresponding measured results to validate the rationality of theoretical solutions.

5.1. Field Test

As shown in Figures 4 and 5, NEPWP and its dissipation in the soil around the pile have been measured previously in field tests of DPC pipe piles with preinstalled pore pressure sensors [9]. These field tests were conducted in Guangzhou. The test site comprised ① mixed fill soil, ② silty silt, ③ muddy clay, ④ silty clay, ⑤ fully weathered, ⑥ strongly weathered conglomerate, and ⑦ moderately weathered conglomerate. The diameter, wall thickness, pile body concrete strength, and reinforcement type of the test piles are 1 m, 130 mm, C80, and AB, respectively. In addition, the diameter and depth of the borehole were 1.02 m and 29 m, respectively. In these field tests, two rows for the pore pressure sensor were set up 1.0 m and 1.5 m from the center axis of piles at depths of 5 m, 10 m, and 20 m.

To reduce error, the sensors are continuously tested to ensure stable readings after installation. Furthermore, the period between the sensor installation and the pile sinking is required to be at least 15 days. As shown in Figure 5, the pore pressure sensor was installed as follows. First, a geological drill bit (130 mm) was used to form the borehole. Second, the sensor was tied and fixed at the corresponding position of the $\phi 16$ reinforcement bar, and then the reinforcement bar and sensor were lifted into the borehole together. Finally, the borehole was filled with fine sand and bentonite balls in a layer-by-layer manner.

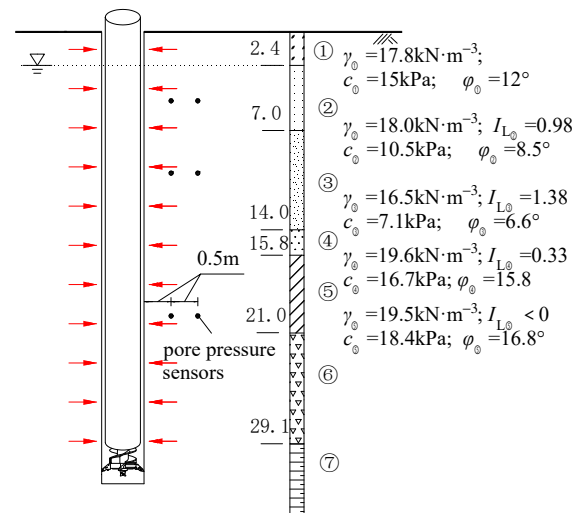


Figure 4. Geological conditions and test schemes.



Figure 5. Pre-buried of pore pressure sensor: (a) securing the pore pressure sensor, (b) inserting the reinforcement bar and sensor, (c) packing fine sand and bentonite balls, and (d) burying the pore pressure sensors along with the reinforcement bar.

5.2. Determination of Initial Pile-Side Soil Pressure

Since DPC pipe piles adopt a pipe pile protection wall, soil pressure coefficients on the pile side differ from the behavior observed in the mud protection wall associated with mud weight [21] and are directly related to soil properties. To characterize the pile-side soil pressure, the coefficient δ is introduced to establish the relationship between the pile-side soil pressure and the initial horizontal total stress, as follows:

$$p_0 = \delta \sigma_{h0}, \quad (20)$$

where δ is the soil pressure coefficient of the pile side. Here, the coefficient δ is determined by employing the presented theoretical solutions to inversely analyze the measured initial pore pressure change data from borehole unloading.

If the coefficient δ is taken as 0.1, 0.2, 0.3, 0.4, and 0.5, then the initial NEPWP after pile sinking is calculated using Equation (8). The calculated parameters, including unit soil weight, cohesive force, and friction angle for each soil layer, are listed in Figure 4. The calculated initial NEPWP at 1.0 m and 1.5 m from the central axis of piles and the corresponding measured values are illustrated in Figure 6. In the silty silt layer with a depth of 5 m, the muddy clay layer with a depth of 10 m, and the fully weathered layer with a depth of 20 m, the measured initial NEPWP values are equivalent to the calculated NEPWP with $\delta = 0.20$ – 0.28 , 0.47 – 0.48 and 0.19 – 0.22 , respectively. Generally, the coefficient δ is taken as a large value of 0.47 – 0.48 for fluid-plastic soils ($I_L > 1$), a small value of 0.19 – 0.22 for hard soils ($I_L \leq 0$), and a linear interpolation for plastic soils ($0 < I_L < 1$). The above results demonstrate that the coefficients δ exhibit significant variability with soil properties owing to the implementation of pipe pile protection wall technology, which differs from the behavior observed in mud protection wall boreholes associated with mud weight [24].

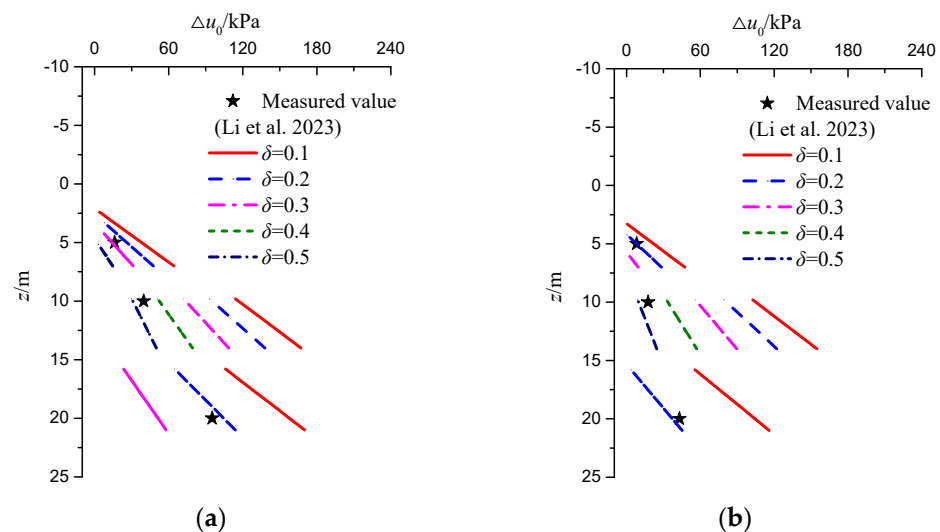


Figure 6. Calculated initial NEPWP and corresponding measured values: (a) $r = 1.0$ m and (b) $r = 1.5$ m [9].

5.3. Model Validation

From the above discussion, the coefficient δ is taken as 0.26, 0.48, and 0.21 at burial depths of 5 m, 10 m, and 20 m, respectively. Then, NEPWP distributions at different moments were calculated using Equations (13)–(16). The consolidation coefficient of the silty silt, muddy clay, and fully weathered layers were $0.1278 \text{ cm}^2/\text{s}$, $0.0012 \text{ cm}^2/\text{s}$, and $0.125 \text{ cm}^2/\text{s}$, respectively. The NEPWP distributions at different moments and the corresponding measured values are shown in Figure 7. As shown, the measured NEPWP dissipation was faster than the corresponding calculated value. It is possible that NEPWP dissipation caused increased soil pressure on the pile side, thereby accelerating the NEPWP dissipation rate synchronously. However, this paper does not consider the coupling effect of

NEPWP dissipation and soil pressure recovery. Overall, the calculated NEPWP distribution curves are consistent with the corresponding measured values with an error range of 5.48–40.08%. These results indicate that the presented theoretical solution can predict the unloading and NEPWP dissipation caused by the sinking of DPC pipe piles.

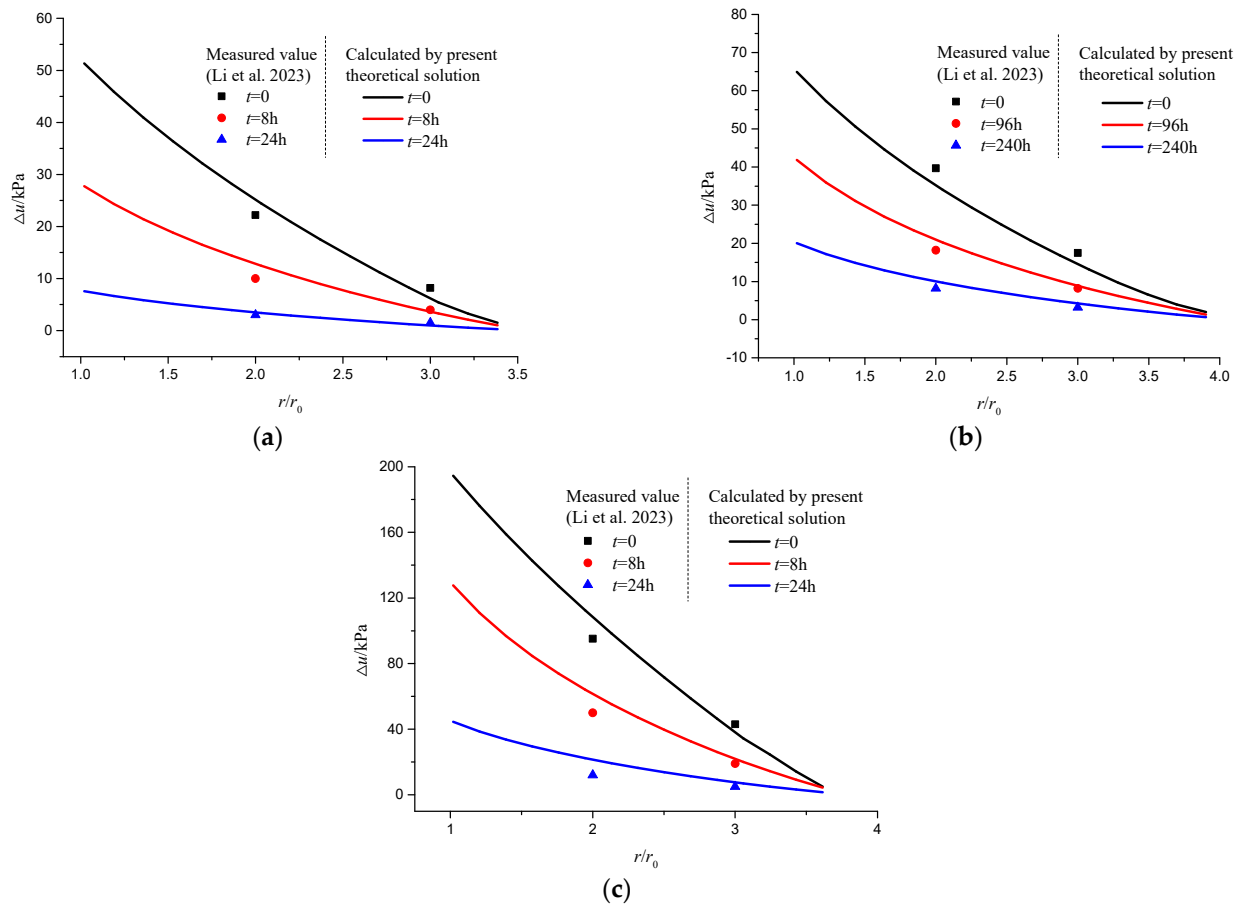


Figure 7. NEPWP distributions compared with corresponding measured values: (a) 5 m buried depth, (b) 10 m buried depth, and (c) 20 m buried depth [9].

6. Impact Factor Analysis

6.1. Effect of Coefficient δ on Unloading

The stress distributions at a depth of 10 m in the soil around piles with different δ are shown in Figure 8. From Figure 8a, it can be seen that the tangential stress along the radial direction first increases and then decreases until converging with the initial horizontal stress. The monotonically increasing and monotonically decreasing regions are the plastic and elastic regions, respectively. In addition, the radial stress increases monotonically until converging with the initial horizontal stress. In the elastic region, the radial stress curve is axisymmetric with the corresponding tangential stress curve, and the sum of these two stresses is constant. In the plastic region, the tangential and radial stresses have consistent monotonicity and decrease with closer proximity to the pile. Accordingly, the average principal stress in this region is less than the initial horizontal total stress, thereby resulting in the formation of NEPWP. In addition, as the value of δ decreases, the radial stress also decreases, and the inflection point of tangential stress becomes correspondingly closer to the right side with a larger plastic region. Figure 8b shows that the distributions of radial and tangential effective stresses exhibit symmetry along the axis of the initial horizontal stress line, demonstrating identical elastoplastic regional properties as those illustrated in Figure 8a.

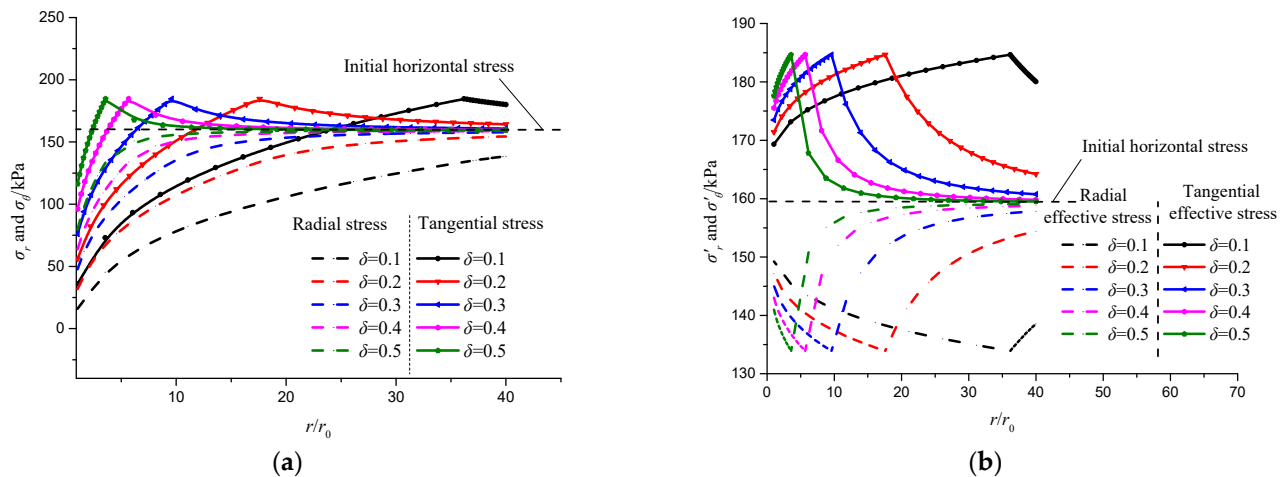


Figure 8. Stress distributions in the soil around piles: (a) Radial and tangential total stress distributions and (b) Radial and tangential effective stress distributions.

Figure 9 shows NEPWP distribution at a depth of 10 m with different δ . As shown, the NEPWP decreases along the radial direction until the elastic–plastic interface tends to zero. The value and distribution range of the NEPWP both increase with the decrease in δ . Thus, δ has a significant influence on the unloading state in the soil around the piles, thereby affecting the subsequent soil pressure recovery on the pile side.

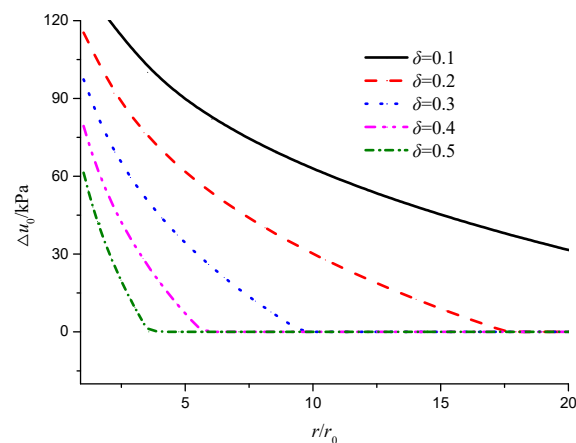


Figure 9. Initial NEPWP distributions in the soil around piles.

6.2. Pile-Side Soil Pressure Recovery

Substituting Equations (8), (13), and (20) into Equation (19), α_{t1} is further expressed as follows:

$$\alpha_{t1} = \frac{1}{\sigma_{h0}} \left[\delta \sigma_{h0} - \frac{(\sigma_{h0} + c \cot \varphi)(\delta \sigma_{h0} - \sigma_{ry})}{\delta \sigma_{h0} + c \cot \varphi} - \sum_{n=1}^{\infty} A_n(r_0) M_0(r_0) e^{-\alpha_n^2 C_{ve} t} \right] \quad (21)$$

From Equation (21), it can be seen that α_{t1} is affected by the coefficients δ , c , φ , and C_{ve} , in addition to being directly related to t . The $\alpha_{t1} - t$ curves with different δ , c , φ , and C_{ve} are illustrated in Figures 10–13. It is clear that α_{t1} exhibits significant changes as t increases. From Equations (17) and (18), it is known that pile-side soil pressure recovery is directly related to α_{t1} . These results indicate that the NEPWP dissipation is a nonnegligible factor contributing to the pile-side soil pressure recovery. The extent of this influence is determined by various coefficients and soil properties.

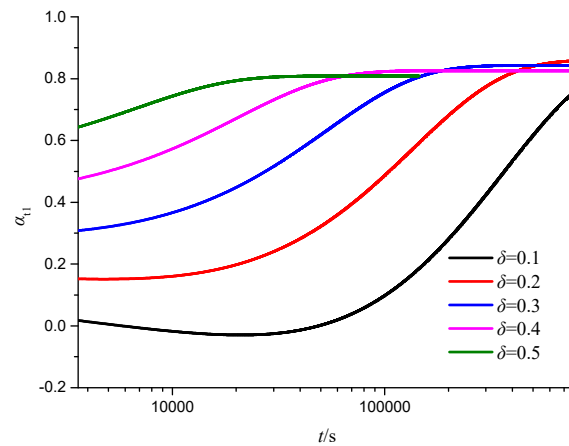


Figure 10. $\alpha_{t1} - t$ curves for different δ values.

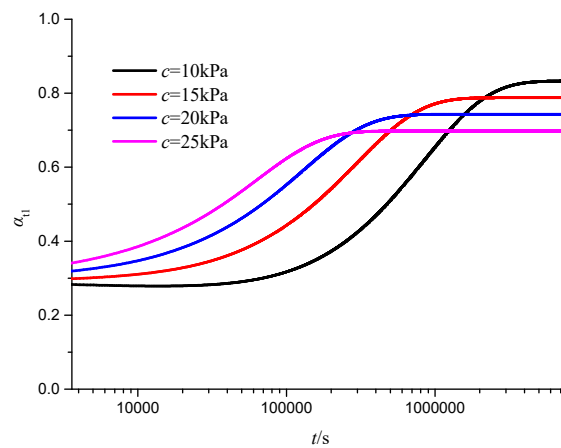


Figure 11. $\alpha_{t1} - t$ curves for different c values.

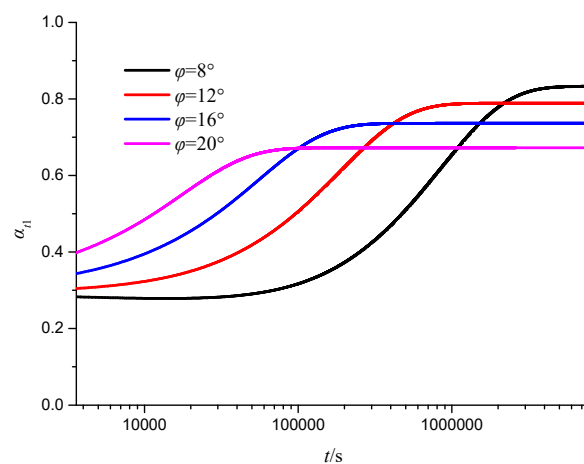


Figure 12. $\alpha_{t1} - t$ curves for different φ values.

Figure 10 shows that the lower bound of α_{t1} varies from 0.10 to 0.49, and the upper bound varies from 0.81 to 0.84, with values of δ from 0.1 to 0.5, indicating that δ mainly affects the lower bound of α_{t1} . The variation range of α_{t1} increases with decreasing values of δ , implying that the influence of NEPWP dissipation is great accordingly. From Figure 11, it can be seen that the lower bound of α_{t1} varies from 0.29 to 0.32, and the upper bound of α_{t1} varies from 0.69 to 0.83, with a value of c from 10 kPa to 25 kPa, which indicates that c mainly affects the upper bound of α_{t1} . As shown in Figure 12, the lower bound of α_{t1}

changes from 0.29 to 0.39, and the upper bound of α_{t1} changes from 0.67 to 0.83, with the value of φ ranging from 8° to 20° , which implies that φ mainly affects the upper bound of α_{t1} , similarly. Moreover, both c and φ have the same influence law on α_{t1} . In fact, if c and φ are large, then the upper bound of α_{t1} is large, but the lower bound is small, indicating that the NEPWP dissipation has a smaller and shorter influence. As can be seen from Figure 13, C_{ve} mainly affects the change rate of α_{t1} and does not affect the upper and lower bounds. Namely, the change rate of α_{t1} decreases with decreasing values of α_{t1} .

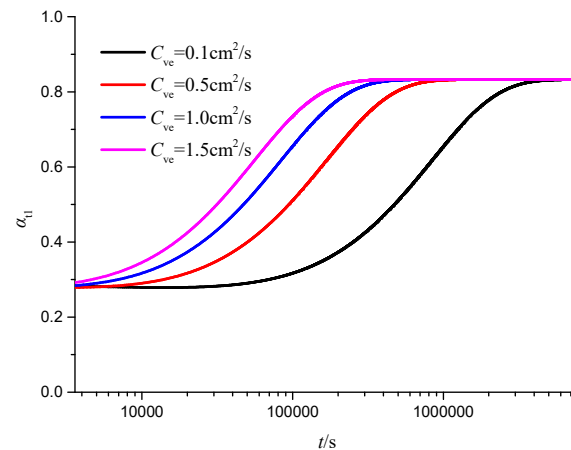


Figure 13. $\alpha_{t1} - t$ curves for different C_{ve} values.

The $\alpha_{t1} - t$ curves for different soil properties are shown in Figure 14, while the detailed calculation parameters can be found in Figure 4. It is evident that, for layers such as silty silt, silty clay, and fully weathered ($I_L \leq 1$) layers characterized by larger values of c , φ , and C_{ve} , the dissipation rate of NEPWP is fast with a shorter influence time, generally not exceeding 1 day. Since the interval between pile sinking and grouting ranges typically from 2 to 7 days, the NEPWP dissipation is completed before grouting operations, which means that the effect of NEPWP dissipation on the grouting can be ignored. Then, the pile-side soil pressure is taken as $p = p_0 + u(r_0, 0)$. Conversely, for muddy clay ($I_L > 1$) layers with smaller values of c , φ , and C_{ve} , the dissipation time of NEPWP exceeds 10 days. Thus, the effect of NEPWP dissipation time on grouting is non-negligible. In such scenarios, it is recommended to perform immediate grouting after pile sinking, mitigating the adverse effects caused by pile-side soil pressure recovery.

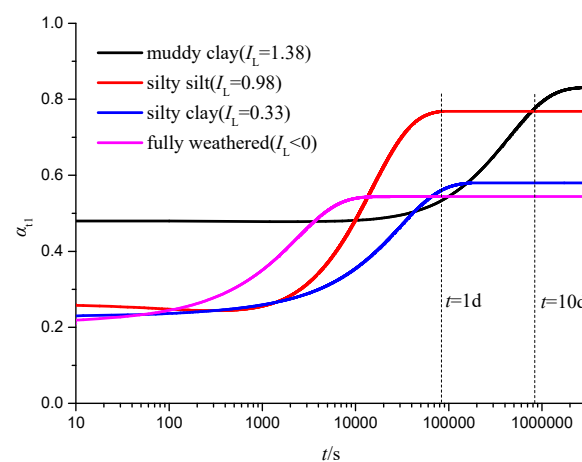


Figure 14. $\alpha_{t1} - t$ curves for different soil properties values.

7. Conclusions

In this paper, theoretical solutions for characterizing unloading and NEPWP dissipation were presented. By inverse-analyzing the measured initial NEPWP data of DPC pipe piles, the initial pile-side soil pressure was determined based on the presented theoretical solutions. Then, the presented theoretical solutions were verified by comparing them with the corresponding measured data. Introducing the time-dependent coefficient, the effect of NEPWP dissipation on the pile-side soil pressure recovery was discussed in detail.

The main conclusions are as follows:

(1) Owing to the implementation of pipe pile protection wall technology, the coefficient δ differs from the behavior observed in mud protection walls associated with mud weight and exhibits significant variability with soil properties, ranging from 0.19 to 0.48. Generally, δ was taken as a large value of 0.47–0.48 for fluid–plastic soils ($I_L > 1$), a small value of 0.19–0.22 for hard soils ($I_L \leq 0$), and a linear interpolation for plastic soils ($0 < I_L < 1$);

(2) NEPWP distribution curves obtained from theoretical solutions are in good agreement with the corresponding measured data, indicating that the proposed approach can effectively predict the unloading induced by the sinking of DPC pipe piles and NEPWP dissipation;

(3) The influence ranges of NEPWP dissipation and soil rheology on the soil pressure recovery on the pile side are $\alpha_{t1} = \delta \sim (p_0 + u_{0,r_0})/\sigma_{h0}$ and $\alpha_{t2} = (p_0 + u_{0,r_0})/\sigma_{h0} \sim 1$, respectively;

(4) NEPWP dissipation is a non-negligible factor contributing to the pile-side soil pressure recovery, and the extent of this contribution is determined by various coefficients and soil properties. δ mainly affects the lower bound of α_{t1} . Both c and φ mainly affect the upper bound of α_{t1} . C_{ve} mainly affects the change rate of α_{t1} , and does not affect the upper and lower bounds;

(5) For plastic ($0 < I_L \leq 1$) and hard ($I_L \leq 0$) soils, NEPWP dissipation is completed before grouting; thus, the effect of NEPWP dissipation on grouting can be ignored, and then the pile-side soil pressure is taken as $p = p_0 + u(r_0, 0)$. In contrast, for fluid–plastic soil ($I_L > 1$), the NEPWP dissipation time has a significant influence on grouting due to the slow NEPWP dissipation. In such scenarios, it is recommended to perform immediate grouting after pile sinking to mitigate adverse effects caused by pile-side soil pressure recovery.

The findings of this research are based on a limited number of field tests and theoretical analyses. The present approach still requires further calibration before it is applied in practical engineering, taking into account factors such as pile–soil interface conditions and soil anisotropy. Furthermore, the effect of soil rheology on the pile-side soil pressure recovery and the impact of pile-side soil pressure recovery on grouting reinforcement remain ambiguous, necessitating further investigation.

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