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WATER AND SOIL PRESSURE CALCULATION METHOD FOR DEEP FOUNDATION PIT WITH SUSPENDED WATERPROOF CURTAIN IN WATER-RICH SANDY COBBLE STRATUM

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ABSTRACT: The water and soil pressure acting on the enclosing structure of a deep foundation pit with a suspended waterproof curtain was studied. Assuming a linear variation of the hydraulic gradient from the bottom of the waterproof curtain to the top of the impermeable layer, a two-dimensional steady-state seepage model was established. The solution for the water head distribution at any point in the seepage field of the foundation pit was obtained using the method of separation of variables. A calculation method for water and soil pressure considering seepage effects was proposed. The verification results indicate that the method proposed in this paper yields a 26.8% larger maximum horizontal displacement of the supporting piles and a 5.2% larger maximum axial force in the internal support compared to the measured values. Furthermore, compared to the classical method, the results of this method are 13.9% and 11.2% lower for these values, respectively. This study fills the gap in the theory of non-complete well seepage and its coupled analysis with water-soil pressure, providing a theoretical breakthrough for the precise design and safety control of deep foundation pits in water-rich strata. The proposed method offers more accurate calculations of key parameters than the classical method, providing a valuable reference for engineering applications and demonstrating practical value.

Keywords: Deep foundation pit; Seepage field solution; Suspended waterproof curtain; Water and soil pressure; Water-rich sandy cobble stratum;

1. INTRODUCTION

Water-rich sandy cobble stratum is widely distributed, characterized by strong permeability and poor stability [1,2], making them highly susceptible to groundwater seepage. The suspended waterproof curtain can extend the seepage path of groundwater, reduce the inflow of water into the foundation pit, and effectively mitigate the impact of groundwater on the surrounding environment of the pit [3]. Therefore, in deep foundation pit projects in water-rich deep and thick sandy cobble stratum, the suspended waterproof curtain is commonly used as an auxiliary measure [4,5]. When dewatering is performed inside the foundation pit with a suspended waterproof curtain in place, groundwater flows into the pit from both the bottom and sides. Due to the influence of the seepage field, the water and soil pressure acting on the waterproof curtain changes accordingly [6].

Accurate calculation of water and soil pressure on the enclosing structure is crucial for the safe construction of foundation pit engineering. However, the calculation of water and soil pressures under the influence of groundwater is relatively complex. Therefore, many experts and scholars have conducted related research. Luo *et al*. (2007) established a differential equilibrium equation based on the balance between the seepage force acting on the soil and the drag force acting on the water, deriving a new formula for calculating the water and soil pressure in foundation pits [7]. It was demonstrated that the seepage force and viscous force are constituted as a pair of action and reaction forces acting on the soil and water, respectively, and that the gravity of the water in the soil is balanced by the viscous force and static pore water pressure. Li (2021), based on the foundation pit engineering of three entrances and exits of Taiyuan Metro Line 2, compared the calculation results of water and soil pressure under the conditions of not considering seepage and considering seepage and found that the calculated values without considering seepage were conservative, which led to increased costs and unnecessary waste [8]. Consequently, it was suggested that seepage should be incorporated into the calculations. Wang (2022) introduced the rock and soil water yield parameter through the principle of force balance, analyzed the water and soil pressure under vertical groundwater seepage and derived a formula for calculating effective stress [9]. Maturi proposed a new method for solving the Laplace equation for steady groundwater flow by combining the Variational Iteration Method (VIM) and the Analytic Solution [10]. Aini *et al*. investigated the influence of changes in pore water pressure under seismic action in sandy soil layers on soil liquefaction potential by combining the coupling method of nonlinear site response analysis and pore water pressure [11]. Shi *et al*. (Xu), focusing on deep foundation pits of subway stations under water-rich conditions, established a 3D seepage-stress coupled model and found that as the dewatering depth increased, the seepage effect in the soil layer significantly intensified, which was detrimental to the stability of the foundation pit [12]. Yuan *et al*. (2012) derived an equation for the groundwater level depression cone curve and studied the total effective stress increment of the foundation pit soil under the combined action of seepage and gravity drainage, obtaining a formula for calculating the effective stress increment [13]. Kui (2014) investigated the interaction between the resistance field and the seepage field, solving the multi-field coupling problem in foundation pits and proposing a new formula for calculating water and soil pressure [14]. Zhang *et al*. (2012) compared the differences in water and soil pressure calculated by separate and integrated methods for silt soil and proposed a method for calculating water pressure in silt while considering the seepage effect [15]. Under the influence of a suspended waterproof curtain, the actual phreatic surface outside the pit is funnel-shaped [16], and the groundwater seepage is classified as unconfined incomplete well seepage. However, from the current research status, current research and practical engineering calculations of water and soil pressure have overlooked the water level drop outside the waterproof curtain [17]. The water-rich and thick sandy cobble strata are characterized by strong permeability and poor stability. Although suspended waterproof curtains are widely used in deep foundation pits in such strata, the complexity of non-complete well seepage and the theoretical deficiencies related to this issue have led to an incomplete solution of the seepage field and an unclear coupling relationship between the curtain seepage and water and soil pressure. Traditional calculation methods, which overlook these core issues, result in significant deviations in key parameters such as pile displacement and internal bracing axial force. This can lead to excessive structural deformation or over-support, which not only restricts design accuracy but also poses safety risks and results in resource waste. Therefore, conducting targeted research to clarify the impact of seepage on water and soil pressure and improve calculation methods has become an urgent need to address practical engineering conflicts.

Based on the current situation, this study, considering the seepage characteristics of the seepage field in deep foundation pits with a suspended waterproof curtain, established a two-dimensional seepage solution model under the assumption that the horizontal hydraulic gradient decreased linearly from the bottom of the waterproof curtain to the top of the impermeable layer. The seepage fields on both the inner and outer sides of the waterproof curtain were solved separately, leading to a method for calculating the water inflow into the foundation pit with a suspended waterproof curtain. Additionally, a formula for determining the water level drawdown at any point outside the waterproof curtain was derived. Based on this, by comprehensively considering the water level drawdown outside the waterproof curtain and the influence of groundwater seepage, a new method for calculating the water and soil pressure on both the active and passive sides of the enclosing structure was proposed.

The subsequent content of the paper is arranged as follows: Chapter 2 presents the theoretical and practical significance of this study; Chapter 3 introduces the theory of water and soil pressure calculation, establishes a two-dimensional seepage model for the foundation pit, and solves the seepage field; Chapter 4 derives the method for calculating water and soil pressure considering seepage effects; Chapter 5 applies the method to the engineering case of the Yangwan Station foundation pit for the Luoyang Metro, verifying the rationality of the calculation methods for water inflow, water level, and water and soil pressure, and conducting a comparative analysis of the displacement of the retaining piles and the axial force of the internal bracing. Finally, the conclusions of the study are summarized, and potential directions for further research are discussed.

2. RESEARCH SIGNIFICANCE

This study introduces a novel approach to analyzing seepage in foundation pits with hanging impermeable curtains in water-rich sandy cobble strata, where traditional methods produce large errors due to incomplete well seepage and unclear seepage–soil-water pressure interactions. The research is original in establishing a dynamic seepage field and proposing a new soil-water pressure calculation method that incorporates seepage effects—filling a clear gap in existing standards. Validated through the Yangwan Station project of Luoyang Metro Line 1, the method significantly improves prediction accuracy. Its applicability to similar strata nationwide highlights its innovative contribution and broad engineering relevance.

3. THE CALCULATION THEORY OF WATER AND SOIL PRESSURE AND THE SOLUTION TO SEEPAGE FIELD IN FOUNDATION PITS

To achieve the goal of calculating soil-water pressure while considering seepage, two key theoretical foundations need to be clarified: first, the calculation principle of soil-water pressure, and second, the method for solving the seepage field. This chapter will first analyze the coupled effects of seepage force and soil gravity based on the theory of soil-water separation, and derive the fundamental expression for soil-water pressure. Then, focusing on the seepage characteristics of a hanging impermeable curtain, a two-dimensional seepage calculation model will be established. The seepage field will be solved using the method of separation of variables, providing the core parameters necessary for the subsequent soil-water pressure calculation.

3.1 Theory of Water and Soil Pressure Calculation

The permeability coefficient of the sandy cobble stratum is relatively large. When calculating the water and soil pressure on the enclosing structure, the contributions of water and soil are considered separately, and the cohesion of the soil layer is ignored. The water and soil pressure acting on the active and passive sides of the enclosing structure can be expressed as:

$$\text{(a) } p_a = K_a\sigma_a + P_{wa} \qquad \text{(b) } p_p = K_p\sigma_p + P_{wp} \tag{1}$$

Where K_a is the active soil pressure coefficient, $K_a = \tan^2(45^\circ - \frac{\varphi}{2})$; K_p is the passive soil pressure coefficient, $K_p = \tan^2(45^\circ + \frac{\varphi}{2})$; φ is the internal friction angle of the soil; σ_a and σ_p are the active and passive soil pressure; P_{wa} and P_{wp} are the active and passive water pressures.

Under steady-state seepage conditions, the groundwater in the surrounding strata of a suspended waterproof curtain foundation pit experiences energy loss due to the blocking effect of soil particles, resulting in a water head loss. In this process, soil particles are subjected to seepage drag, also known as seepage force, while the water unit experiences a viscous force of equal magnitude but in the opposite direction. The effect of groundwater seepage on the self-weight stress of soil is presented in Fig. 1.

For the active side of the enclosing structure, which corresponds to the exterior of the foundation pit, the seepage force acts vertically downward, aligning with the direction of the soil's self-weight stress. Conversely, the seepage force on the passive side acts in the opposite direction. The expressions for active and passive soil pressures under seepage influence are expressed as follows:

$$\text{(a) } \sigma_a = G_a + j_a \qquad \text{(b) } \sigma_p = G_p + j_p \tag{2}$$

Where G_a and G_p represent the self-weight stress of the soil on the active side and passive side, respectively, j_a and j_p represent the seepage forces acting on the active and passive sides of the soil, respectively, and are directly influenced by the groundwater seepage hydraulic gradient.

Therefore, to accurately obtain the water pressure, the groundwater seepage hydraulic gradient, and the water level outside the waterproof curtain on both the active and passive sides of the enclosing structure, it is essential to solve the seepage field of the foundation pit.

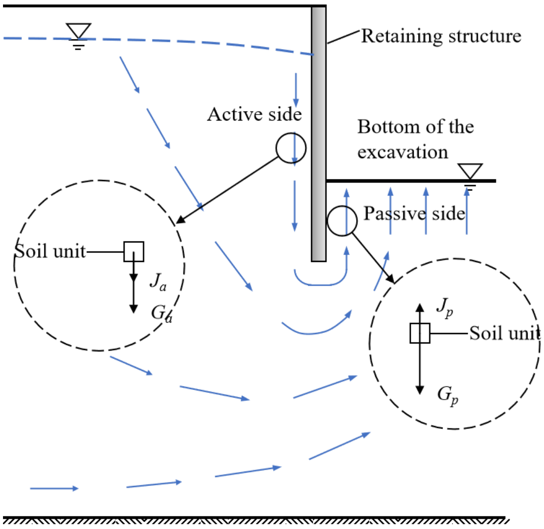
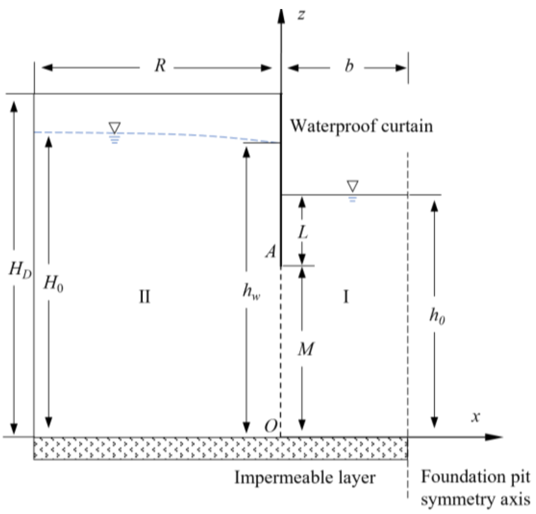


Fig. 1 Schematic diagram of groundwater seepage

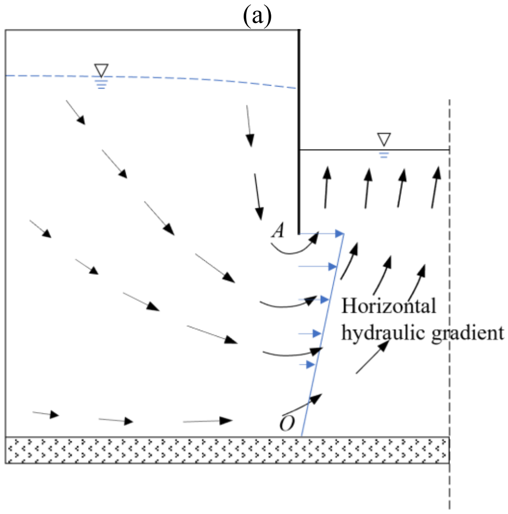
3.2 Suspended Foundation Pit Seepage Calculation Model

The research object was selected as a strip-shaped foundation pit, and it was assumed that the strata in which the pit was located were homogeneous media and that groundwater seepage was isotropic. For the strip-shaped foundation pit, the groundwater seepage along a section parallel to the pit width could be regarded as two-dimensional seepage. Additionally, due to the symmetry of the foundation pit seepage, only half of the section was considered for the study. Owing to the fact that the width of the waterproof curtain was much smaller than the pit dimensions and for the convenience of solving the foundation pit seepage field, the width of the waterproof curtain was not considered in the establishment of the calculation model. The intersection of the extension line of the waterproof curtain and the impermeable layer was chosen as the coordinate origin, and the coordinate system shown in Fig. 2(a) was established. Fig. 2(b) is the schematic diagram of the seepage field.

In Fig. 2(a), h_0 is the water level in the foundation pit; h_w is the water level outside the waterproof curtain; L is the depth of the suspended waterproof curtain inserted into the water level in the foundation pit; M is the thickness of the water-crossing section; b is half of the foundation pit width; R is the dewatering influence radius; H_0 is the water level at the influence radius; H_D is the thickness of the cover layer. All the above parameters are in meters (m).



(a)



(b)

Fig. 2 Foundation pit seepage model: (a) Seepage field calculation model, (b) Seepage field of foundation pit with suspended waterproof curtain

It was assumed that the groundwater seepage in the soil of the foundation pit complied with Darcy's law. According to Groundwater dynamics [20], the seepage of the strata satisfies the steady-state seepage equation. The seepage field of the foundation pit was divided into two parts, I and II, along the extension line of the waterproof curtain, and the water head distribution functions of the two regions were $H_1(x, z)$ and $H_2(x, z)$ respectively.

The water head at the foundation pit's influence radius (x=-R) and the groundwater level inside the foundation pit (z=h_0) remain constant. The boundary conditions are defined as follows:

$$\begin{cases} x=-R,\ 0\le z\le H_0,\ H_2=H_0 \\ z=h_0,\ 0\le x\le b,\ H_1=h_0 \end{cases} \tag{3}$$

The top of the impermeable layer (z = 0) and the sides of the waterproof curtain (x = 0, $M \le z \le L$) are impermeable boundaries. According to the symmetry of the seepage, there is no horizontal seepage at the symmetric axis of the foundation pit (x=b), which can also be treated as an impermeable boundary. The boundary conditions are as follows:

$$\begin{cases} z=0,\ -R\le x\le b,\ \dfrac{\partial H_1}{\partial z}=0 \\ x=0,\ M\le z\le h_0,\ \dfrac{\partial H_{1,2}}{\partial x}=0 \\ x=b,\ 0\le z\le h_0,\ \dfrac{\partial H_1}{\partial x}=0 \end{cases} \tag{4}$$

The position of the phreatic water table level outside the waterproof curtain is uncertain. According to the characteristics of the actual foundation pit seepage field, the total water head value on phreatic water table level gradually decreases from the influence radius to the waterproof curtain, which can be recorded as $\dfrac{\partial H_2}{\partial x}\Big|_{z=H_0}<0$. On the other hand, the groundwater outside the waterproof curtain flows toward the water-crossing section, creating vertical downward seepage at the surface of the phreatic layer, with the maximum seepage velocity occurring at the waterproof curtain, which can be denoted as $\dfrac{\partial H_2}{\partial z}\Big|_{z=H_0}>0$.

In the seepage field of the suspended waterproof curtain in a deep foundation pit, under the effect of pumping inside the foundation pit, groundwater from both sides of the foundation pit seeps into the pit through the water-crossing section at the bottom of the waterproof curtain. Foundation pits using suspended waterproof curtains are typically deep, and the water-crossing section has a large thickness. Due to the limited effect of pumping in the pit, as the foundation pit depth increases, the groundwater seepage velocity decreases. If the thickness of the aquifer is much greater than the depth of the waterproof curtain inserted into the aquifer, the groundwater velocity at the top of the impermeable layer approaches zero (Li et al. 2024). Based on the above, it was assumed that the horizontal hydraulic gradient in the region from the bottom of the waterproof curtain to the top of the impermeable layer (x=0, 0≤z≤M) decreased linearly to zero. Therefore:

$$x=0,\ 0\le z\le M,\ \frac{\partial H_{1,2}}{\partial x}=J_x(z) \tag{5}$$

Assuming that the horizontal hydraulic gradient distribution function J_x(z) of the OA section is: $J_x(z)=kz$, according to Darcy's law:

$V_x(z) = -KJ_x(z)$, where K was the permeability coefficient of the stratum, m/d. By integrating the horizontal velocity of the OA section, which equaled the water inflow per unit width from one side of the foundation pit, the following relationship could be derived:

$$\int_0^M V_x(z)dz = \int_L^M -KJ_x(z)dz = Q \tag{6}$$

After solving, the horizontal seepage gradient distribution function J_x(z) of the OA section could be obtained as:

$$J_x(z) = -\frac{2Q}{KM^2} z \ (0 \le z \le M) \tag{7}$$

3.3 Solution of Seepage Field in Region I

According to the foundation pit seepage calculation model, the governing equation for solving the total water head distribution function in Region I was [20]:

$$\frac{\partial^2 H_1}{\partial x^2} + \frac{\partial^2 H_1}{\partial z^2} = 0 \tag{8}$$

Using the method of separation of variables, it is assumed that the solution to Eq. (8) can be expressed as $H_1(x,z) = X(x)Z(z) + C_0$, which can be treated as solving the following system of Eq. (9)

$$\begin{aligned} &\text{(a) } X''(x) - \lambda X(x) = 0 \\ &\text{(b) } Z''(z) + \lambda Z(z) = 0 \end{aligned} \tag{9}$$

To satisfy the upper boundary condition of Region I, the value of C_0 should be h_0, according to the upper and lower boundary conditions of Region I, it could be organized as: $\begin{cases} X(x)Z(h_0) = 0 \\ X(x)Z'(0) = 0 \end{cases}$, and it could be solved: $\begin{cases} Z(h_0) = 0 \\ Z'(0) = 0 \end{cases}$.

Combining the above conclusions to solve Eq. (9), it was found that there was a solution only when λ >0. At this point, the eigenvalue $\lambda_n = \left[\frac{(2n-1)\pi}{2h_0}\right]^2$, the eigenvalue equation $Z_n(z) = \cos\frac{(2n-1)\pi}{2h_0} z$, let $C_n = \frac{(2n-1)\pi}{2h_0}$, $n = 1, 2, 3, ...$. According to the principle of superposition, the general solution for the head distribution function in region I was Eq. (10)

$$H_1(x,z) = h_0 + \sum_{n=1}^{\infty}(A_n e^{C_n x} + B_n e^{-C_n x})\cos C_n z, n = 1,2,3,... \tag{10}$$

The right boundary condition of Region I ($\frac{\partial H_1}{\partial x}|_{x=b}$ =0) could be used to find: $B_n = A_n e^{2C_n b}$, and according to the left boundary condition ($\frac{\partial H_1}{\partial x}|_{x=0} = \begin{cases} -\frac{2Q}{KM^2} z & (0 \le z \le M) \\ 0 & (M < z \le h_0) \end{cases}$), as well as the orthogonality of the Fourier series, we could obtain: $A_n = -\frac{4Q}{h_0 KM^2}\frac{(C_n M \sin C_n M + \cos C_n M - 1)}{C_n^3(1-\mathrm{e}^{2C_n b})}$. Thus, the water head distribution function of Region I could be organized as Eq. (11).

$$H_1(x,z) = h_0 - \frac{4Q}{h_0 KM^2} \times \sum_{n=1}^{\infty}\left\{\begin{aligned} &\frac{(C_n M \sin C_n M + \cos C_n M - 1)}{C_n^3(1-\mathrm{e}^{2C_n b})} \times \\ &[e^{C_n x} + e^{C_n(2b-x)}]\cos C_n z \end{aligned}\right\}$$
$$\begin{cases} 0 \le x \le b \\ 0 \le z \le h_0 \end{cases}$$
(11)

3.4 Solution of Seepage Field in Region II

The solution process for Region II is similar to that of Region I. The method of separation of variables is also applied to solve the water head distribution function in Region II. To satisfy the left boundary condition of Region II ($H_2|_{x=-R} = H_0$), it was assumed that $H_2(x,z) = H_0 + X(x)Z(z)$, and according to the lower boundary condition ($\frac{\partial H_2}{\partial z}|_{z=0}$ =0), $Z_n(z) = \cos C_n' z$ could be obtained.

Based on the superposition principle, the general solution of the water head distribution function in Region II was Eq. (12).

$$H_2(x,z) = H_0 + \sum_{n=1}^{\infty}(A_n' e^{C_n' x} + B_n' e^{-C_n' x})\cos C_n' z \quad n = 1,2,3,... \tag{12}$$

The theoretical dewatering influence radius of the foundation pit is infinitely large. However, to satisfy the left boundary condition of Region II ($H_2|_{x=-R} = H_0$), the value of B_n' should be 0. Then, the general solution for the water head distribution function in Region II was:

$$H_2(x,z) = H_0 + \sum_{n=1}^{\infty} A_n^{'} e^{C_n^{'} x} \cos C_n^{'} z, n = 1,2,3,... \quad (13)$$

According to the right boundary condition $\frac{\partial H_2}{\partial x}\Big|_{x=0} = \begin{cases} -\frac{2Q}{KM^2} z & (0 \le z \le M) \\ 0 & (M < z \le H_0) \end{cases}$, as well as the orthogonality of the Fourier series, it could be found as: $A_n^{'} = -\frac{4Q}{H_0 KM^2} \frac{(C_n^{'} M \sin C_n^{'} M + \cos C_n^{'} M - 1)}{C_n^{'3}}$.

The value of $C_n^{'}$ in the above equation is determined by the upper boundary condition. In practical foundation pit dewatering, the upper boundary condition of Region II is relatively complex, and it is unreasonable to simply regard it as a fixed water head boundary ($H_2|_{z=-R} = H_0$)[13,19]. For a fixed water head boundary, the corresponding $C_n^{'}$ value is: $\frac{(2n-1)\pi}{2H_0}, n = 1,2,3,...$. Based on previous discussions regarding the phreatic surface boundary, it must satisfy $\frac{\partial H_2}{\partial x}\Big|_{z=H_0} < 0$, $\frac{\partial H_2}{\partial z}\Big|_{z=H_0} > 0$. After trial calculation, taking $C_n^{'} = \frac{(2n-1)\pi}{4H_0}, n = 1,2,3,...$ satisfies the requirements, and the water head distribution function of Region II was formulated as Eq. (14).

According to the continuity condition of groundwater seepage, the water head distribution at the adjacent boundary of the two regions must satisfy their respective distribution functions, that was $H_1\Big|_{\substack{x=0 \\ 0 \le z \le M}} = H_2\Big|_{\substack{x=0 \\ 0 \le z \le M}}$. By substituting the adjacent boundary points into the water head distribution functions of the two regions and combining them, an expression for the water inflow on one side of the foundation pit could be derived.

$$H_2(x,z) = H_0 - \frac{4Q}{H_0 KM^2} \times \sum_{n=1}^{\infty} \frac{(C_n^{'} M \sin C_n^{'} M + \cos C_n^{'} M - 1)}{C_n^{'3}} e^{C_n^{'} x} \cos C_n^{'} z \quad (14)$$
$$\begin{cases} -\infty \le x \le 0 \\ 0 \le z \le H_0 \end{cases}$$

For computational convenience, the intersection of the extended line of the waterproof curtain and the top of the impermeable layer (x=0, z=0) was chosen as the calculation point. Through simultaneous solution, the following equation was obtained as Eq. (15).

$$Q = \frac{H_0 - h_0}{\frac{4}{H_0 KM^2} \times E^{'} - \frac{4}{h_0 KM^2} \times F^{'}}$$
$$E^{'} = \sum_{n=1}^{\infty} \frac{(C_n^{'} M \sin C_n^{'} M + \cos C_n^{'} M - 1)}{C_n^{'3}} \quad (15)$$
$$F^{'} = \sum_{n=1}^{\infty} \frac{(C_n M \sin C_n M + \cos C_n M - 1)}{C_n^3 (1 - e^{2C_n b})} (1 + e^{2C_n b})$$

There is no pressure water head on the phreatic water table level outside the waterproof curtain, and the position water head at the phreatic water table level represents the total water head. That is, $H_2(x,z) = z$. By substituting this equation into the water head distribution function outside the waterproof curtain, the expression for the water level drawdown at any point outside the waterproof curtain could be derived as Eq. (16).

$$z = H_0 - \frac{4Q}{H_0 KM^2} \times \sum_{n=1}^{\infty} \frac{(C_n^{'} M \sin C_n^{'} M + \cos C_n^{'} M - 1)}{C_n^{'3}} e^{C_n^{'} x} \cos C_n^{'} z \quad (16)$$

When x=0 in Eq. (16), the corresponding z value represents the water level outside the waterproof curtain (h_w).

4. CALCULATION METHOD OF WATER AND SOIL PRESSURE

4.1 Calculation of Water Pressure

The total water head of groundwater consists of position water head, pressure water head, and velocity water head [20], as expressed by the following Eq.(17).

$$H = z + \frac{v^2}{2g} + H_w \quad (17)$$

The pressure water head acting on the passive side of the enclosing structure can be expressed as Eq. (18).

$$H_{wp} = H_1(0,z) - z - \frac{v_p^2}{2g} \quad (18)$$

To explicitly represent the water pressure at a depth of h_p from the groundwater level, let: $z = h_0 - h_p$. Then, the expression for the water pressure distribution at a depth of h_p from the groundwater level on the passive side of the enclosing structure can be obtained as Eq. (19).

$$P_{wp}=\gamma_w\left\{\begin{array}{l}h_0-\dfrac{4Q}{h_0KM^2}\times\\ \sum_{n=1}^{\infty}\left[\begin{array}{l}\dfrac{(C_nM\sin C_nM+\cos C_nM-1)}{C_n^3(1-e^{2C_nb})}\\ \times(1+e^{2C_nb})\cos(C_n(h_0-h_p))\end{array}\right]\\ -(h_0-h_p)-\dfrac{v_p^2}{2g}\end{array}\right\} \quad (19)$$
$$0\le h_p\le(h_0-M)$$

Where v_p is the groundwater seepage velocity at the calculation point on the passive side. Similarly, the water pressure distribution expression at a depth of h_a from the ground surface on the active side of the enclosing structure can be obtained as Eq. (20).

$$P_{wa}=\gamma_w\left\{\begin{array}{l}H_0-\dfrac{4Q}{H_0KM^2}\times\\ \sum_{n=1}^{\infty}\left[\begin{array}{l}\dfrac{(C_n^{'}M\sin C_n^{'}M+\cos C_n^{'}M-1)}{C_n^{'3}}\\ \times\cos(C_n^{'}(H_D-h_a))\end{array}\right]\\ -(H_D-h_a)-\dfrac{v_a^2}{2g}\end{array}\right\} \quad (20)$$
$$\left(H_D-h_w\right)\le h_a\le(H_D-M)$$

Where v_a is represents the groundwater seepage velocity at the calculation point on the active side.

4.2 Calculation of Soil Pressure under Seepage Influence

For the passive side of the enclosing structure, the groundwater seeps upward vertically, and the seepage force is opposite to the direction of the soil self-weight stress. Since the groundwater level is roughly level with the bottom of the foundation pit, the elevation of the two is regarded as equal, the vertical stress of the soil element at a depth of h_p from the bottom of the foundation pit is:

$$\sigma_p=\gamma' h_p+j_p \qquad 0\le h_p\le(h_0-M) \quad (21)$$

Where j_p is the seepage force of unit soil at h_p from the passive side to the bottom of the foundation pit. The seepage force acting on a soil element at a given point is calculated as the product of the hydraulic gradient at that point and the unit weight of water. It is important to note that the quantity to be determined is the total seepage force exerted on the soil mass above the depth of h_p from the bottom of the foundation pit. This value is obtained by integrating the seepage forces acting on all soil elements from the excavation base down to the depth of h_p. The mathematical expression of j_p is given as follows:

$$j_p=\int_0^{h_p}\gamma_w i_p d_{h_p} \quad (22)$$

Where i_p represents the calculation of the seepage hydraulic gradient at a depth of h_p from the bottom of the foundation pit, $i_p=\left.\dfrac{\partial H_1}{\partial z}\right|_{\substack{x=0\\ z=h_0-h_p}}$.

Based on the seepage theory in Chapter 3, the vertical stress correction formula is given by the following Eqs. (23) and (24):

$$\sigma_a=\gamma h_a \qquad 0\le h_a\le\left(H_D-h_w\right) \quad (23)$$

$$\sigma_a=\gamma(H_D-h_w)+\gamma'(h_a-(H_D-h_w))+j_a \qquad \left(H_D-h_w\right)<h_a\le(H_D-M) \quad (24)$$

Where γ' is the buoyant unit weight of the soil; j_a is the seepage force per unit soil element at the depth of h_a from the ground surface on the active side, and its value is:

$$j_a=\int_{H_D-h_w}^{h_a}\gamma_w i_a d_{h_a} \quad (25)$$

Where i_a is the seepage hydraulic gradient at that point, $i_a=\left.\dfrac{\partial H_2}{\partial z}\right|_{\substack{x=0\\ z=H_D-h_a}}$.

To perform a trial calculation of the total head distribution function in the foundation pit seepage field, it is noticed that the hydraulic gradient i_a from the water level at the active side of the enclosing structure (x = 0, z = h_w) to the bottom of the waterproof curtain (x = 0, z = M) and the hydraulic gradient i_p from the water level at the passive side of the enclosing structure (x = 0, z = h_0) to the bottom of the waterproof curtain (x = 0, z = M) change slightly. To simplify the calculation process and facilitate engineering applications, the average hydraulic gradients $i_a^{'}$ and $i_p^{'}$ for the two seepage sections are used in the calculations.

4.3 Calculation of Water and Soil Pressure under Seepage Influence

Groundwater flows from regions of higher hydraulic head to those of lower head. However, groundwater seeps upward from the bottom of the foundation pit, indicating that the hydraulic head H_1 decreases with increasing depth z; thus, parameter i_p is less than zero. Conversely, on the active side, groundwater flows downward, resulting in a positive value for parameter i_a. This implies a reduction in earth pressure on the passive side and an increase in soil pressure on the active side, which is consistent with the actual situation.

The seepage force on the soil at a depth h_p from the bottom of the foundation pit on the passive side is given by:

$$j_p = \gamma_w i_p^{'} h_p \tag{26}$$

The seepage force on the soil at a depth h_a from the ground surface on the active side is given by:

$$j_a = \gamma_w i_a^{'} [h_a - (H_D - h_w)] \tag{27}$$

By simultaneously solving Eqs. (1) (2) (20) (23) (24) and (27), the water and soil pressure acting on the active side of the enclosing structure could be obtained as Eq. (28).

Similarly, by simultaneously solving Eqs. (1) (2) (19) (21) and (26), the water and soil pressure acting on the passive side of the enclosing structure could be obtained as Eq. (29).

Since the actual groundwater seepage velocity is relatively slow, typically a few millimeters per second, the velocity head is much smaller than the pressure head and elevation head and can, therefore, be neglected in calculations.

$$p_a = \begin{cases} K_a \sigma_a; \\ 0 \le h_a \le \left(H_D - h_w\right) \\ K_a \sigma_a + P_{wa}; \\ \left(H_D - h_w\right) < h_a \le (H_D - M) \end{cases}$$

$$= \begin{cases} K_a \gamma h_a; \\ 0 \le h_a \le \left(H_D - h_w\right) \\ K_a[\gamma(H_D - h_w) + \gamma'(h_a - (H_D - h_w)) \\ +\gamma_w i_a^{'}(h_a - (H_D - h_w))] \\ +\gamma_w \left\{ \begin{array}{l} H_0 - (H_D - h_a) - \dfrac{v_a^2}{2g} - \dfrac{4Q}{H_0 K M^2} \times \\ \displaystyle\sum_{n=1}^{\infty} \left[\begin{array}{c} \dfrac{(C_n^{'} M \sin C_n^{'} M + \cos C_n^{'} M - 1)}{C_n^{'3}} \\ \times \cos(C_n^{'}(H_D - h_a)) \end{array} \right] \end{array} \right\} \\ \left(H_D - h_w\right) < h_a \le (H_D - M) \end{cases} \tag{28}$$

$$\begin{aligned} p_p &= K_p \sigma_p + P_{wp} \\ &= K_p(\gamma' h_p + \gamma_w i_p^{'} h_p) + \\ &\gamma_w \left\{ \begin{array}{l} h_0 - (h_0 - h_p) - \dfrac{v_p^2}{2g} - \\ \dfrac{4Q}{h_0 K M^2} \displaystyle\sum_{n=1}^{\infty} \left[\begin{array}{c} \dfrac{(C_n M \sin C_n M + \cos C_n M - 1)}{C_n^3 (1 - \mathrm{e}^{2C_n b})} \\ \times (1 + e^{2C_n b}) \cos(C_n (h_0 - h_p)) \end{array} \right] \end{array} \right\} \\ &0 \le h_p \le (h_0 - M) \end{aligned} \tag{29}$$

5. ENGINEERING VERIFICATION

5.1 Project Overview

Fig. 3 The stratum exposed in the foundation pit of Yangwan Station

The groundwater type at the terminal station of Luoyang Metro Line 1, Yangwan Metro Station, is classified as Quaternary pore phreatic water, primarily stored in the sandy cobble stratum with strong permeability. The exposed strata are shown in Fig. 3. The total length of the station is 432.3 m, with a standard section width of 20.7 m and a depth of approximately 18 m. The foundation pit supporting form consists of interlocking piles with internal support. According to the geological survey report for Yangwan Station, the groundwater level is located 10 meters below the ground surface, the aquifer thickness is 90 m, and the permeability coefficient of

Table 1. Parameter values of the foundation pit

Cover layer thickness H_D/m	Aquifer thickness H_0/m	Water level in the pit h_0/m	Stratum permeability coefficient K/m/d	Foundation pit width $2b$/m	Thickness of water-crossing section M/m
100	90	81.62	60	20.7	72

Table 2. Comparison of measured and calculated values

	Measured value	Calculated value
the water inflow per unit width Q/m^3/m · d	78.44	94.14
Water level outside the waterproof curtain h_w/m	85.92	87.77

the stratum is high, approximately 60 m/d.

Cn April 28, 2018, water pumping operations were conducted at the west end of the foundation pit of Yangwan Station. The depth of this section of the foundation pit is 18.38 m, with a support depth of 28 m and a water-crossing section thickness of 72 m. Under stable pumping conditions, the water level in the pit decreased by 8.38 m compared to the initial groundwater level, stabilizing at 81.62 m. During the pumping process, the the water inflow per unit width on one side of the foundation pit was measured at 78.44 m³/m·d. Additionally, the water level in the observation well outside the waterproof curtain dropped by 4.08 meters compared to the initial level, resulting in a water level of 85.92 m outside the waterproof curtain.

5.2 Verification of Water Inflow and Water Level outside the Waterproof Curtain

Based on the project overview, the parameter values used in the calculation model are listed in Table 1.To verify the rationality of the calculation methods for water inflow and groundwater level outside the waterproof curtain, it is necessary to measure both the water inflow and the groundwater level.

Substituting the parameters listed in Table 1 into Eq. (14) allowed for the calculation of the the water inflow per unit width Q on one side of the foundation pit. Subsequently, by substituting the obtained water inflow per unit width and the parameters listed in Table 2 into Eq. (15), the water level h_w outside the waterproof curtain could be determined. Due to the complexity of the calculation process, MATLAB software was employed for computation. After several trial calculations, it was found that when the number of terms n is set to 10, the variation in water head values between the two regions is less than 0.01%. Therefore, setting n=100 satisfies the required precision for the calculations. The comparison between the calculated and measured results is shown in Table 2.

The calculated value of the the water inflow per unit width for the foundation pit is 94.14 m³/m·d, which is 20.02% larger than the measured value of 78.44 m³/m·d. The calculated value of the water level outside the waterproof curtain is 87.77 m, which is 2.15% larger than the measured value of 85.92 m.

Currently, the seepage theory for foundation pit with a suspended waterproof curtain is not sufficiently developed, and the existing code (MHURDPRC 2012) does not provide a specific calculation method for the water inflow of a foundation pit with a suspended waterproof curtain. Consequently, the water inflow used in the design stage often involves relatively errors. According to engineering experience, an error of 20.02% can still meet the engineering requirements. The calculation error in the water level outside the waterproof curtain is relatively small and meets the engineering requirements.

Both calculation errors fall within the allowable range for engineering applications, indicating that the calculated results for water inflow and water level outside the waterproof curtain for a strip foundation pit with a suspended waterproof curtain, as proposed in this paper, are reasonable and effective.

5.3 Calculation of Water and Soil Pressure

Currently, in the engineering practice, the calculation of water and soil pressure on the enclosing structure of the foundation pit often neglects the influence of groundwater seepage and assumes the groundwater level outside the waterproof curtain to be horizontal. As a result, the final calculated values deviate from the actual conditions. Based on the proposed method for calculating water and soil pressure, the water and soil pressure acting on the enclosing structure of the suspended waterproof curtain at Yangwan Station is calculated. At the same time, referring to the actual engineering conditions of the Yangwan Station, the parameters of the foundation pit are selected according to the values in Table 1, and the water inflow and water level outside the waterproof curtain are taken from the calculated values in Table 2. The stratum where Yangwan Station is located primarily consists of a medium-dense gravel layer, with a cohesion of zero. The stratum parameters are listed in Table 3.

To comparatively validate the effectiveness of the water and soil pressure calculation method proposed in this study, both the classical method and the proposed method were used to calculate the water and soil pressure acting on the enclosing structure of the Yangwan Station foundation pit. The calculation results of both methods are shown in Fig. 4.

Under the influence of seepage, soil particles experience drag forces due to groundwater movement, while the water units are subjected to viscous forces of equal magnitude in the opposite direction. The seepage force on the active side is downward, while on the passive side, it is opposite. This causes the active side soil pressure to increase and the water pressure to decrease, while the passive side soil pressure decreases and the water pressure increases. The classical method for calculating water and soil pressure does not consider the effects of seepage and assumes that the external groundwater level is horizontal. Engineering measurements have shown that this assumption leads to an overestimation of the calculated results (Dou et al. 2017). In contrast, the results from the calculation method proposed in this paper are slightly smaller than those obtained using the classical method, making them more aligned with actual conditions.

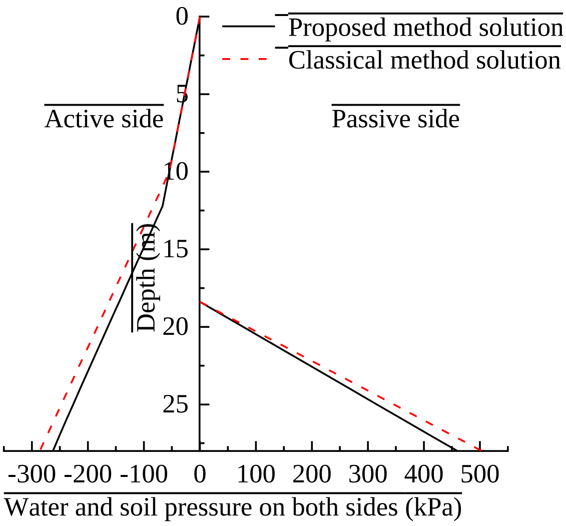


Fig. 4 Distribution curve of water and soil pressure on the enclosing structure

Furthermore, field measurements indicate that the water level outside the curtain has dropped by 4.08 meters compared to the initial water level. Existing methods, which assume no drop in the water level, result in an overestimation of the active-side water pressure. According to Eq. (20), water pressure is directly related to the water level elevation. Ignoring the 4.08-meter drop leads to a discrepancy of approximately 40.8 kPa in the water pressure calculation at a given depth on the active side, causing the calculation of the total active-side water and soil pressure to be somewhat conservative. This discrepancy is then carried over into the calculation of the supporting structure, causing the stress predictions to deviate from actual conditions.

5.4 Calculation and Comparison of Maximum Horizontal Displacement of Piles and Internal Support Axial Force

In engineering design, water and soil pressure serve as a crucial basis for the design of internal supports. The elastic support method (MHURDPRC 2012) is employed to calculate the internal support axial force corresponding to two sets of water and soil pressures, which are then compared with the actual measurement data of the foundation pit and analyzed. The foundation pit enclosing structure of Yangwan Station uses bored piles with internal supports, with three internal supports installed. The first support is an 800×900 concrete support, spaced 9 m apart and 2 m deep from the ground. The second and third supports are Ø609, t=16 mm steel pipe supports, spaced 3 m apart, with depths of 9 m and 14 m from the ground, respectively.

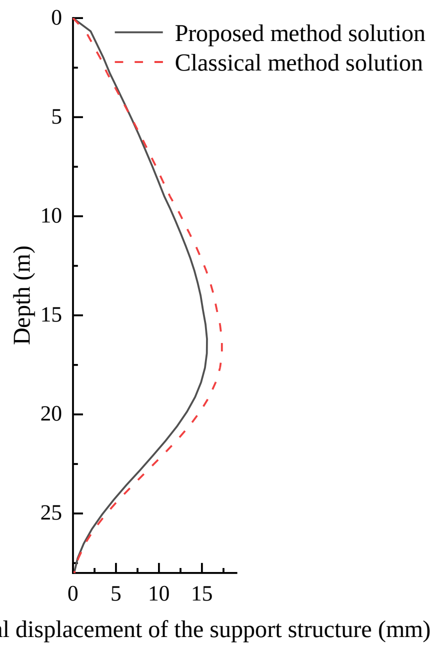


Fig. 5 Lateral displacement curve of the enclosing structure

Table 3. Parameter values for water and soil pressure calculation

Unit weight of groundwater γ_w / kN/m³	Unit weight of soil layer γ / kN/m³	Buoyant unit weight of soil layer γ' / kN/m³	Internal friction angle of soil layer φ / °
10	21	11	36

Table 4. Displacement of internal support points

	First support /mm	Second support /mm	Third support /mm
Proposed method solution	4.3	11.3	15.1
Classical method solution	4.0	12.1	16.7

Table 5. Comparison of measured and calculated values of internal support axial force

	First support / kN	Second support / kN	Third support / kN
Measured value	892.0	1858.5	2899.0
Proposed method solution	997.6	2282.6	3050.2
Classical method solution	928.0	2444.2	3373.4

Since the Yangwan Station foundation pit has multiple supports and the load distribution pattern is complex, the finite element numerical calculation software ANSYS (2022.R1) is used for solving the model. During model establishment, the parameters are selected in accordance with code regulations. The elastic moduli of the concrete material and the steel support are 30GPa and 210GPa respectively; their densities are 2500 kg/m³ and 7850 kg/m³, respectively; and the Poisson's ratios are 0.1668 and 0.3, respectively. Considering the horizontal spacing of the supports, the elastic modulus of the supports is adjusted based on the spacing during model establishment. By applying the two sets of water and soil pressures previously obtained to the model, the horizontal displacement curve of the support piles can be derived, as shown in Fig. 5.

The maximum horizontal displacement of the piles corresponding to this solution is 15.6 mm, while the value for the classical solution is 17.3 mm. The maximum displacement occurs near the foundation pit excavation face. The actual measured maximum horizontal displacement of the piles is 12.3 mm, with the classical method's calculation result being relatively more conservative.

The elastic stiffness coefficient of the internal support is multiplied by the corresponding position horizontal displacement distance to obtain the internal support axial force. According to the current code (MHURDPRC 2012), the elastic stiffness coefficient K_i (kN/m) of the support within the calculation width can be computed using the following formula:

$$K_i = EA / \lambda Sl \tag{30}$$

Where E is the elastic modulus of the support material, Pa; A is the cross-sectional area of the support member, m²; λ is the support immovable point adjustment coefficient. When the soil layer properties, depths, etc. of the two sides of the support are similar, and the excavation is symmetrically layered, it is taken as 0.5; l is the calculation length of the horizontal support member, m; S is the spacing of the horizontal support members, m.

The elastic moduli of the concrete and steel supports are 30 GPa and 210 GPa, respectively. The cross-sectional areas are 0.72 m² and 0.0298 m², respectively. The spacings of the horizontal support members are 9 m and 3 m, respectively, and the horizontal calculation lengths are all 20.7 m.

According to Eq. (30), the elastic stiffness coefficients of the first reinforced concrete support is: $K_1 = 2.32\times10^{\wedge}5\ \mathrm{kN/m}$, and the second and third steel supports are: $K_2 = K_3 = 2.02\times10^{\wedge}5\ \mathrm{kN/m}$. The displacement of the support points corresponding to the two models is extracted as shown in Table 4.

The internal support axial force is obtained by multiplying the displacement of the support points in Table 4 with the corresponding internal support elastic stiffness coefficients. The actual support axial force value is measured using an axial force meter. The comparison of the calculated and measured values of the internal support axial force is shown in Table 5.

The axial force of the internal support calculated by the proposed water and soil pressure solution and the classical method are both larger than the measured values. For the first internal bracing, the results from the proposed solution and the classical method are 11.8% and 4.0% larger than the measured values, respectively; for the second internal bracing, the deviations are 22.8% and 31.5%, respectively; for the third internal bracing, the deviations are 5.2% and 16.4%, respectively.

The actual data measurements are influenced by many accidental factors, and the theoretical calculation also has significant limitations. Therefore, an inevitable difference exists between the calculation results and the measured values. From the comparison of the internal support axial forces, the axial force values corresponding to the water and soil pressure solution in this paper are closer to the measured values, indicating that this method is more reliable than the classical approach. The first support is located above or near the groundwater level, where the seepage influence is minimal. Therefore, the error in the traditional calculation method is relatively small. However, the second and third supports are typically located within areas of high groundwater permeability, where water and soil pressures are significantly influenced by dynamic water conditions. Therefore, in water-rich and highly permeable strata, the water and soil pressure calculation method proposed in this study shows promising potential for application.

6. CONCLUSION

To obtain the distribution of water and soil pressure on the foundation pit enclosing structure under the influence of groundwater seepage, this study, based on the Yangwan Station foundation pit project, conducts the theoretical analysis and provides a solution for a strip foundation pit with a suspended waterproof curtain in thick aquifers. The following conclusions were drawn:

1. According to the seepage characteristics of the suspended waterproof curtain foundation pit in the deep aquifer, the foundation pit seepage field was divided into two regions. It was assumed that the horizontal water hydraulic gradient from the bottom of the waterproof curtain to the top of the impermeable layer decreased linearly. The seepage field was solved using the method of separation of variables and the principle of superposition, resulting in the derivation of the water head distribution function for the seepage area of the foundation pit.
2. Based on the water head distribution function in the seepage area of the foundation pit, the calculation expressions for the the water inflow per unit width of the strip foundation pit with a suspended waterproof curtain and the water level outside the waterproof curtain were further derived. The effectiveness of these calculations was verified by comparing them with the actual measurement data from the Yangwan Station foundation pit.
3. Based on the solved seepage field of the foundation pit, a novel method for calculating the water and soil pressure on the enclosing structure of the suspended waterproof curtain foundation pit was proposed. The calculation results were compared with those from the classical method and with on-site actual measurement data, demonstrating that the water and soil pressure calculation results in this study are reasonable and reliable.

This study developed a model for solving the stable seepage field in suspended deep foundation pits within water-rich, thick sandy cobble strata, enriching the theoretical framework for calculating key parameters of suspended deep foundation pits. This model holds significant practical engineering value. However, the assumption of a linear variation in the horizontal hydraulic gradient may lead to deviations in the calculation results under different engineering conditions. Additionally, abrupt changes in the permeability coefficient within the actual strata can impact the applicability of the proposed method. Therefore, future research will focus on analyzing the problem under conditions that more closely resemble real-world situations, aiming to provide a theoretical foundation for foundation pit engineering in deeper and more complex environments.

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