

Article

Research on Application of High-Pressure Cyclic Grouting Technology in Soft Soil Layers

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Abstract

Aiming at technical challenges such as the insufficient bearing capacity of orifice formation leading to slurry overflow and non-uniform formation reinforcement in soft soil layer grouting engineering, an external cyclic grouting process through the grouting pipe is innovatively proposed. Distinguished from traditional in-hole circulation methods, this process achieves bottom-up cyclic grouting through a slurry return channel outside the grouting hole, which effectively reduces the risk of orifice fracturing and improves grouting uniformity. A grouting pressure loss equation is established to quantitatively analyze the relationships between the allowable grouting pressure and the side wall opening of the grouting pipe, slurry rheological parameters, surface consolidation depth, and surface consolidation strength. It is revealed that slurry with high viscosity and low yield stress is suitable for deep grouting, and a design criterion innovatively proposes that the side wall opening of the grouting hole should dynamically increase with the grouting depth. Based on the strain–pressure curve, a prediction model for the reinforcement radius of compaction grouting is established. Slurry rheological parameters and formation mechanical parameters are obtained through laboratory tests, and field grouting tests are conducted. The reinforcement effect is verified by means of ground-penetrating radar and standard penetration tests. The results show that, compared with traditional grouting processes, this process significantly improves the bearing capacity of the orifice and enhances the uniformity and compactness of formation reinforcement and that the theoretical prediction error of the reinforcement radius is less than 15%. The research results provide the theoretical basis and technical support for soft soil grouting engineering and have important engineering application value.

Keywords: cyclic grouting; soft soil reinforcement; field test; rheological parameters



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1. Introduction

Grouting reinforcement technology for soft soil formations is a critical research area in geotechnical engineering. However, its practical application has long been limited by numerous challenges associated with special geological conditions. Soft soils are characterized by a high void ratio, low permeability coefficient, and weak structural strength, which lead to two core engineering problems: (1) sealing failure is likely to occur, in which slurry easily overflows along the casing during high-pressure grouting, thereby limiting the potential for pressure enhancement; and (2) slurry diffusion is non-uniform, as slurry

tends to flow randomly along preferential channels, resulting in leakage in some areas and rendering precise control of the reinforcement range difficult.

Regarding the issue of orifice sealing failure, Zhang et al. [1–3] developed a high-pressure sealing slurry and proposed a pulsating grouting method, achieving a pressure of 8.0 MPa at a depth of 120 m. However, orifice overflow still occurred at higher pressures, revealing an upper limit to the bearing capacity of the sealing material. Hong [4] pointed out that in static pressure grouting within soft soil layers, excessive pressure leads to shear failure of the soft soil at the orifice. Liu et al. [5], through field investigations of five coal mines, found that packer capsule rupture mostly occurs in soft rock formations, indicating that the insufficient bearing capacity of the soft rock is the root cause of sealing failure. These studies suggest that the upper limit of grouting pressure is essentially governed by the shear strength of the soil surrounding the orifice rather than by the strength of the sealing material itself. Regarding the problem of non-uniform slurry diffusion, Ouyang et al. [6] and Li et al. [7] noted that slurry tends to flow preferentially along weak surfaces or large pores, resulting in a diffusion distance difference of 3–5 times between preferential and non-preferential channels. Hu et al. [8] established a theoretical model for slurry diffusion in inclined tubular fractures, and sensitivity analysis showed that factors such as fracture width, grouting pressure, and rheological coefficients have significantly different impacts on preferential versus non-preferential diffusion. Tang et al. [9] demonstrated that soil heterogeneity exacerbates the irregularity of diffusion. Although the diffusion mechanism is well understood, effective technical means to achieve uniform diffusion are still lacking.

In terms of pressure loss prediction, Zhang et al. [10] and Sun et al. [11] established diffusion equations based on the Herschel–Bulkley rheological model, revealing the quantitative relationship between pressure loss and diffusion distance, as well as the key role of yield stress. Li et al. [12] demonstrated that slurry pressure distribution is highly sensitive to gap size and rheological parameters. Zhang et al. [13] found that rheological parameters directly affect slurry diffusion distance and pressure loss. Gao et al. [14], Tang et al. [15], and Li et al. [16] systematically analyzed the influence mechanism of rheological parameters on pressure loss through numerical simulation. However, these models mostly focus on single fractures and do not consider the special geometric condition of the annular gap on the outer wall of the grouting pipe, lacking a systematic method coupled with reinforcement range design.

Regarding soil response and reinforcement effect prediction in soft soil grouting, Ye et al. [17] established a coupled compaction–fracturing diffusion model, finding that the maximum diffusion distance has an exponential relationship with the grouting pressure difference. Zhang et al. [18] studied reinforcement effects based on nonlinear compaction. Wu et al. [19] established an analytical model considering the interaction between grouting pressure and soft clay. Liu et al. [20] investigated the spatiotemporal evolution law of grouting pressure in soft soil through scaled model tests. Li et al. [21] revealed the pressure transmission mechanism and soil stress redistribution law of multi-position grouting. Jin et al. [22] optimized the slurry rheological properties and construction parameters for synchronous grouting in soft soil shield tunnels. These studies provide important references for understanding soil response, but the models often consider too many factors, resulting in complex formulas and insufficient applicability.

Based on the aforementioned problems, this study proposes a novel external cyclic grouting technology for soft soil formations. Unlike traditional in-hole circulation methods, this technology utilizes a slurry return channel outside the grouting pipe. The slurry is injected into the bottom grouting section and then rises along the annular gap between the pipe and the borehole wall to achieve cyclic grouting, fundamentally altering the

pressure distribution pattern of traditional grouting. A grouting pressure loss equation is established to quantitatively analyze the coupling relationships between the allowable grouting pressure and the side wall opening, consolidation depth, consolidation strength, and slurry rheological parameters. This reveals the mechanism by which slurry with high viscosity and low yield stress is suitable for deep grouting. Based on this equation, a design criterion is innovatively proposed where the side wall opening should dynamically increase with depth to ensure uniform pressure distribution. Additionally, a prediction model for the reinforcement radius of compaction grouting is established based on the strain–pressure curve, providing a theoretical basis for the grouting hole spacing design. Slurry rheological parameters and formation mechanical parameters are obtained through laboratory tests, and field grouting tests are conducted. The reinforcement effect is verified using ground-penetrating radar and standard penetration tests. The results show that, compared with traditional grouting processes, this technology effectively improves the bearable pressure at the orifice and significantly enhances the compactness and uniformity of the soil after grouting.

As shown in Figure 1, prior to grouting, the surface formation is pretreated. The perforated pipe grouting method is employed to drill and grout, thereby increasing the fracture initiation pressure P_f of the surface formation. The pre-embedded pipe has a burial depth of L_1 , and its radius R_1 corresponds to the borehole radius. The grouting pipe has a radius of R_2 , and the grouting hole length is L with a grouting section length of L_2 . After the bottom grouting section is filled, the slurry flows upward along the outer wall of the grouting pipe to perform cyclic grouting on the formation. The borehole radius R_1 should be slightly larger than the grouting pipe radius R_2 to minimize the annular gap between the borehole wall and the grouting pipe, thereby increasing slurry flow resistance while avoiding an excessively small gap that could impede slurry circulation.

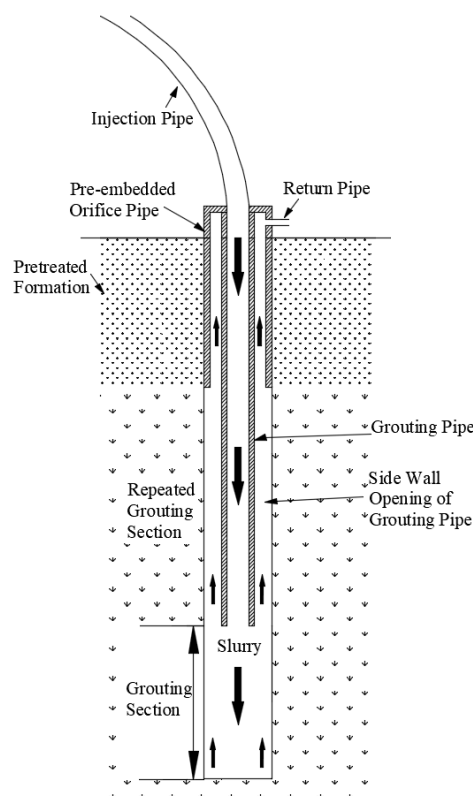


Figure 1. Schematic diagram of high-pressure cyclic grouting.

The average velocity in the annular region can be obtained as:

$$\bar{v} = \frac{h^2 dp}{3\mu dl} - \frac{h\tau_0}{2\mu} \quad (3)$$

$$q_1 = 4\pi h(R_2 + h)\bar{v} \quad (4)$$

Thus:

$$\frac{dp}{dl} = \left(\frac{3\tau_0}{2} \frac{1}{h(l)} + \frac{3q_1\mu}{4\pi} \frac{1}{h(l)^3} \frac{1}{R_2 + h(l)} \right) \quad (5)$$

By solving the simultaneous equations, the pressure loss Δp_1 can be obtained as:

$$\Delta p_1 = \frac{3\tau_0 L}{2h_0(n-1)} \ln n + \frac{3Lq_1\mu}{4\pi h_0(n-1)} \left[\frac{n^2-1}{2n^2 R_2 h_0^2} - \frac{n-1}{n R_2^2 h_0} + \frac{\ln n}{R_2^3} + \frac{1}{R_2^3} \ln \left(\frac{R_2 + h_0}{R_2 + n h_0} \right) \right] \quad (6)$$

where when $n = 1$, i.e., under the condition of constant opening:

$$\Delta p_1 = \frac{3\mu L}{h_0^3} \left(\frac{q_1}{4\pi(h_0 + R_2)} + \frac{\tau_0 h_0^2}{2\mu} \right) \quad (7)$$

Under the condition that channels exist on the outer wall surface of the grouting pipe, the slurry preferentially flows along these channels. However, during the grouting process, a situation of low injection rate q_2 may occur. In this case, it is necessary to close the return valve or increase the injection rate to raise the grouting pressure and ensure the injection rate in the grouting section. At this time, fracturing is prone to occur at the lower end of the pre-embedded pipe. When the grouting pressure at the lower end of the pre-embedded pipe exceeds the fracturing pressure, fracturing occurs. It is assumed that the fracturing is annular and that the fracture opening b is uniform. There is an inverse relationship between the fracture opening and the fracture initiation pressure; the greater the fracture initiation pressure, the smaller the fracture opening b .

The pressure loss during the fracturing process is similar to the pressure loss in an annular pipe, which can be obtained as:

$$\Delta p_2 = P_f + \frac{24\mu L_1}{b^3} \left(\frac{q_1}{2\pi(2R_1 + b)} + \frac{\tau_0 b^2}{8\mu} \right) \quad (8)$$

Therefore, the allowable grouting pressure can be obtained as follows:

$$P = \Delta p_1 + \Delta p_2 \quad (9)$$

When the grouting pressure exceeds this value, seal failure occurs, and slurry overflow along the outer wall surface of the pre-embedded pipe will appear.

In addition, to ensure that the slurry has sufficient grouting pressure during circulation along the side wall opening of the grouting pipe to repeatedly grout the formation, it is necessary to avoid excessive pressure dissipation during the circulation process. The concept of flow pressure loss ratio λ is proposed, namely:

$$\frac{\Delta p_1}{\Delta p_1 + \Delta p_2} = \lambda \quad (10)$$

where $0 < \lambda < 1$. The larger λ is, the greater the flow pressure loss and the more difficult it is for the slurry to effectively re-consolidate the grouted area. Therefore, λ needs to be below

a certain value to ensure the working stability and grouting effectiveness of the grouting circulation system.

2.3. Theoretical Model of Reinforcement Radius

The nonlinear compression characteristics of formations are a core research concept in geotechnical engineering, manifested as the nonlinear relationship between compressive deformation and stress. As stress increases, the compression modulus of the formation gradually decreases; the deformation rate accelerates as well, which is mainly caused by factors such as soil particle rearrangement, pore water expulsion, and plastic deformation. The ε - P curve intuitively reflects the compression behavior of the formation by plotting the relationship between strain and effective stress. Kondner first proposed a hyperbolic equation to describe the relationship between soil compression strain and pressure. Its model form is simple with few parameters and is widely applied. The functional relationship is expressed as:

$$\varepsilon = \frac{P}{E_0 + mP} \quad (11)$$

where ε is the compression strain of the soil, P is the compressive stress, and E_0 and m are the characteristic parameters of the function. The intercept E_0 is the reciprocal of the initial compression modulus, and m is a material parameter that controls the degree of nonlinearity of the compression curve. Both parameters can be obtained through experiments.

It is assumed that during the grouting process, the slurry diffuses uniformly along the grouting section and the formation is uniformly compressed. Thus:

$$\frac{Q - \pi R_1^2 L_2}{\pi L} = R^2 \quad (12)$$

where L_2 is the grouting section length, Q is the injection volume, ignoring the slurry diffusion volume in the gap on the outer wall surface of the grouting pipe, R_1 is the borehole radius, and R is the slurry diffusion radius.

Combining Equations (11) and (12), the solution equation for the grouting reinforcement radius R_c under different grouting pressures can be established:

$$R_c = \sqrt{\left(\frac{Q - \pi R_1^2 L_2}{\pi L} \right) \left(\frac{E_0}{P} + m \right)} \quad (13)$$

3. Analysis of Influencing Factors on Grouting Pressure

In analyzing the influence of different factors on the allowable grouting pressure, the focus is on analyzing the effects of the side wall opening of the grouting pipe, slurry yield stress, slurry viscosity, surface reinforcement depth, and surface fracture initiation pressure on the allowable grouting pressure based on the following given parameters: the grouting section length L ranges from 5 m to 30 m, the grouting pipe radius R_2 is 5 cm, the borehole diameter R_1 is 6 cm, the side wall opening of the grouting pipe h_0 is 0.5 cm, the baseline value of n is 4, the surface fracture initiation pressure P_f is 1.0 MPa, the orifice pipe burial depth L_1 is 1.0 m, the slurry injection rate q_0 is 15.0 L/min, the slurry plastic viscosity μ is 2.0 Pa·s, the yield stress τ_0 is 50 Pa, and the fracture opening b at seal failure is 0.2 cm.

3.1. Influence of Slurry Characteristics on Allowable Grouting Pressure and Flow Pressure Loss Ratio

The variation curves of the allowable grouting pressure with slurry yield stress and viscosity are shown in Figure 3. Analysis reveals that the allowable grouting pressure exhibits a significant positive correlation with slurry viscosity and yield stress. Under

30 m depth conditions, when slurry viscosity increases from 0.5 Pa·s to 5 Pa·s, the allowable pressure increases from 2.3 MPa to 7.9 MPa, an increase of 3.4 times, with a growth slope of allowable pressure reaching 1.2 MPa/(Pa·s). In contrast, when yield stress varies within the range of 20~200 Pa, the pressure increase at the same depth is only 1.4 times, with a growth slope of 0.014 MPa/Pa. The enhancement efficiency of viscosity increase on grouting pressure is 85.7 times that of yield stress, indicating that the allowable grouting pressure is more sensitive to viscosity changes.

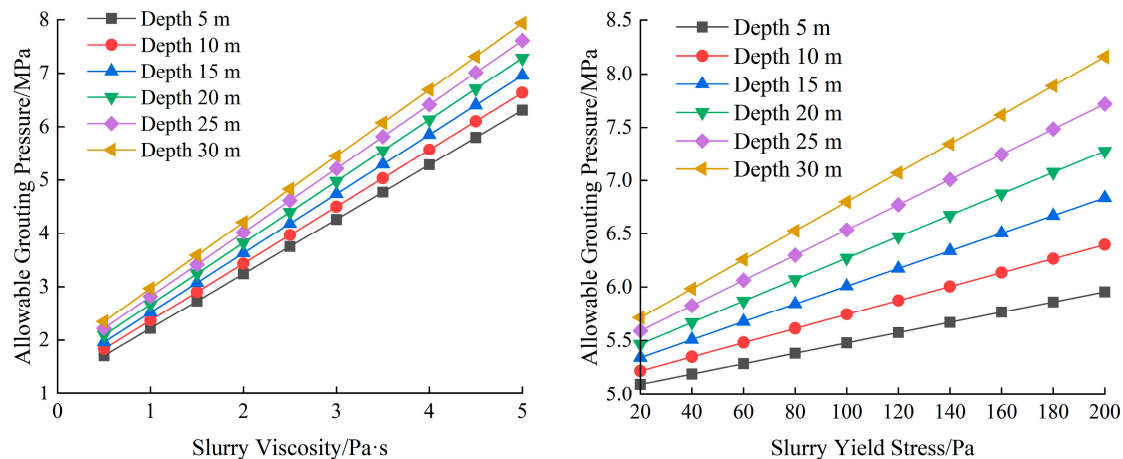


Figure 3. Variation curves of allowable grouting pressure with yield stress and viscosity.

The influence of slurry yield stress and viscosity on the allowable grouting pressure gradually amplifies with increasing formation depth. For example, at a depth of 5 m, every 10 Pa increase in yield stress increases the allowable pressure by 0.05 MPa, while at a depth of 30 m, every 10 Pa increase in yield stress increases the allowable pressure to 0.14 MPa. At a depth of 5 m, the viscosity contribution to the allowable pressure increase is 1.0 MPa/(Pa·s), while at a depth of 30 m, it increases to 1.2 MPa/(Pa·s).

The variation curves of flow pressure loss ratio λ with slurry yield stress and viscosity are shown in Figure 4. Analysis reveals that when slurry viscosity increases from 0.5 Pa·s to 5 Pa·s, λ decreases from 0.33 to 0.25 under 30 m depth conditions, a reduction of 25%. The reason is that high-viscosity slurry leads to a significant increase in pressure loss during the fracturing process, while the increase in pressure loss during slurry flow along the side wall opening of the grouting pipe is smaller, resulting in a decrease in the flow pressure loss ratio. When yield stress increases from 20 Pa to 200 Pa, λ increases significantly, indicating that the increase in yield stress raises the flow pressure loss ratio.

When slurry yield stress increases from 20 Pa to 200 Pa, the pressure loss ratio increases significantly, but the increase magnitude decreases with increasing grouting depth: at a depth of 5 m, the pressure loss ratio increases by 3.1 times; at a depth of 30 m, the increase magnitude decreases to 2.5 times. In contrast to yield stress, when slurry viscosity increases from 0.5 Pa·s to 5 Pa·s, the pressure loss ratio shows a decreasing trend, and the decrease magnitude gradually diminishes with increasing depth: at a depth of 5 m, the pressure loss ratio decreases by 31%; at a depth of 30 m, the decrease narrows to 25%. Combined with Equation (8), it can be concluded that fracture pressure loss is more sensitive to viscosity changes.

In summary, the increase in slurry yield stress raises the allowable grouting pressure but tends to result in an excessive pressure loss ratio λ , while the increase in slurry viscosity raises the allowable grouting pressure and simultaneously reduces the pressure loss ratio λ . To ensure cyclic grouting effectiveness and to avoid an excessive pressure loss ratio, slurry

yield stress should not be too high. For deep grouting especially, grouting materials with high viscosity and low yield stress should be prioritized.

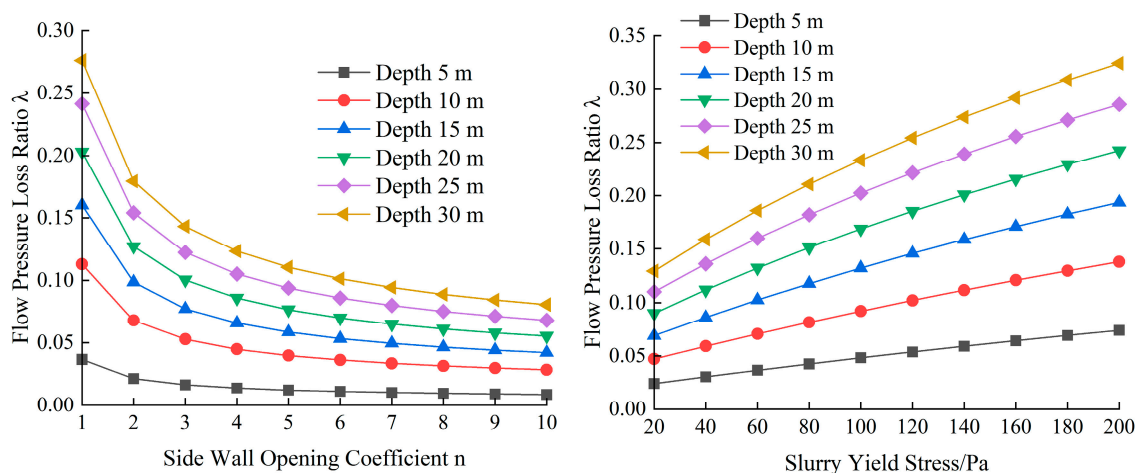


Figure 4. Variation curves of flow pressure loss ratio λ with yield stress and viscosity.

3.2. Influence of Side Wall Opening Variation of Grouting Pipe on Allowable Grouting Pressure and Flow Pressure Loss Ratio

Analysis of Figure 5 reveals that the allowable grouting pressure exhibits a significant negative correlation with the side wall opening of the grouting pipe. As the opening increases from 0.2 cm to 2.0 cm, the allowable grouting pressure P at different depths decreases overall by 50.5%~80.1%, indicating that larger flow channels significantly reduce slurry flow resistance. The attenuation process exhibits two-stage characteristics: a rapid attenuation stage with an opening ranging from 0.2 cm to 0.6 cm, where at a 30 m depth, the allowable grouting pressure drops sharply from 13.3 MPa to 3.8 MPa. In this stage, for every 0.1 cm increase in the opening, the allowable pressure attenuates by 2.4 MPa, a decrease of 17.8%; a stable attenuation stage with an opening ranging from 0.6 cm to 2.0 cm, where in this stage, for every 0.1 cm increase in the opening, the decrease in the allowable pressure slows to 2.2%. At an opening of 0.2 cm, the allowable grouting pressure at a 30 m depth is 2.7 times that at a 5 m depth, while at an opening of 2.0 cm, this ratio decreases to 1.1 times, indicating that as the opening increases, the influence of depth on the allowable grouting pressure gradually diminishes.

The pressure loss ratio exhibits nonlinear attenuation with an increasing opening. At a 30 m depth, in the low opening region of 0.2 cm~0.6 cm, λ decreases from 0.76 to 0.22; in the high opening region of 0.6 cm~2.0 cm, λ decreases from 0.22 to 0.09, eventually stabilizing in the range of 0.09~0.12. This indicates that as the opening increases, λ first decreases rapidly and then tends to stabilize. Additionally, as the depth increases, the critical opening threshold changes significantly. When λ is 0.1, the corresponding opening value at a 5 m depth is 0.4 cm, while the minimum opening value at a 30 m depth is 2.0 cm, a 5-fold difference in opening requirements.

By fitting the relationship curve between the side wall opening of the grouting pipe h_0 and the grouting section length L when λ is 0.1, Figure 6 can be obtained. Where $h_0 = 0.279 \times 10^{0.063L}$ the fitting coefficient R^2 is 0.995, indicating a high degree of fit. The formula indicates that when using this process for cyclic grouting, it is necessary to consider the interaction between formation depth and the side wall opening of the grouting pipe. As depth increases, the opening should gradually increase to meet the pressure requirements of cyclic grouting.

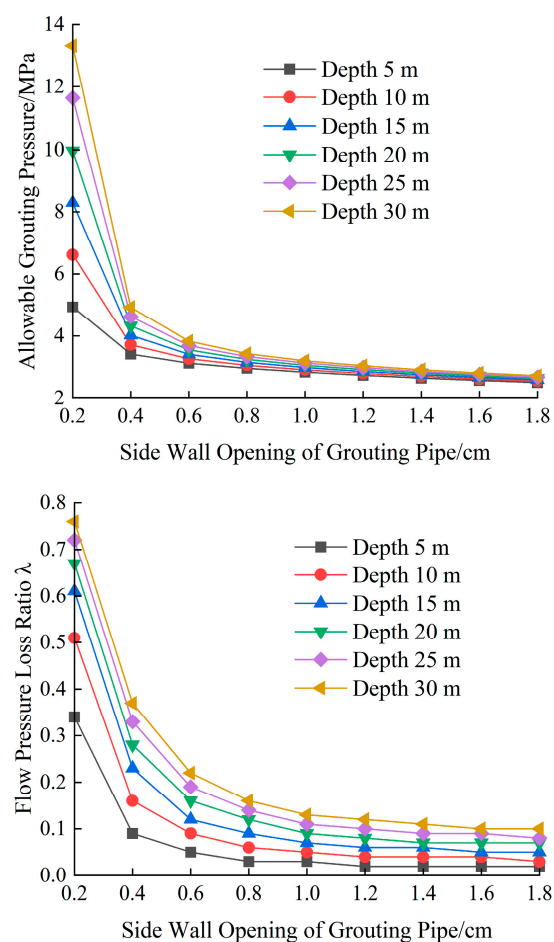


Figure 5. Variation curves of allowable grouting pressure and flow pressure loss ratio with side wall opening of grouting pipe.

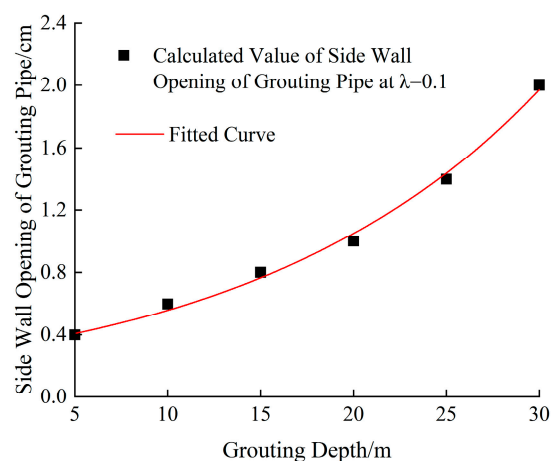


Figure 6. Variation curve of side wall opening of grouting pipe when flow pressure loss ratio λ is 0.1.

Analysis of Figure 7 reveals that at different depths, the allowable grouting pressure monotonically decreases with an increasing opening coefficient, but the attenuation gradient varies with grouting depth. At a grouting depth of 30 m, when n increases from 1 to 10, P decreases by 21.2%; at a depth of 5 m, the comparable decrease is 4.6%. This indicates that as depth increases, the influence of n variation on the allowable grouting pressure P becomes increasingly significant. This pattern suggests that during deep grouting, special attention should be paid to the influence of n variation on the allowable grouting pressure. By reducing grouting pressure while improving the reinforcement strength of the orifice

pipe to avoid seal failure, the efficiency and safety of the grouting process can be achieved. λ exhibits nonlinear attenuation with increasing n , and the attenuation rate decreases with increasing depth. At a grouting depth of 30 m, when n increases from 1 to 10, λ decreases by 70.8%; at a depth of 5 m, the comparable decrease is 76.4%.

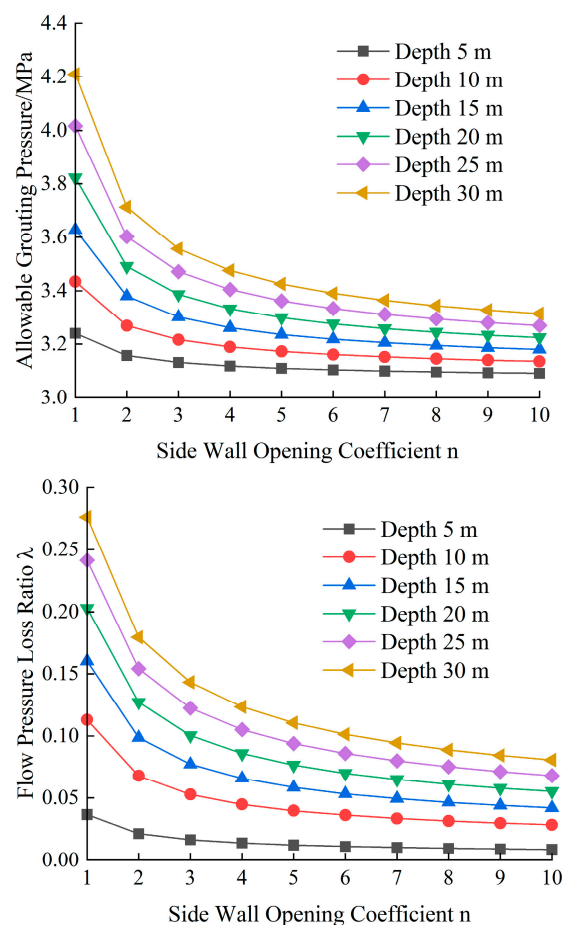


Figure 7. Variation curves of allowable grouting pressure and flow pressure loss ratio λ with side wall opening coefficient n of grouting pipe.

In actual engineering, the variation in the gap channel opening coefficient n essentially reflects a certain degree of deformation of the soil surrounding the re-grouting section during the grouting process. If the variation in n is ignored, the deviation rate between the calculated and actual values of the allowable grouting pressure will reach 4.6%–21.2%, which may potentially lead to a fracturing risk.

3.3. Influence of Surface Consolidation on Allowable Grouting Pressure and Flow Pressure Loss Ratio

The variation curves of the allowable grouting pressure and the flow pressure loss ratio with surface consolidation depth are presented in Figure 8. Analysis reveals that the allowable grouting pressure increases significantly and linearly with increasing consolidation depth. For every 20% increase in consolidation depth, the allowable grouting pressure increases by 15%–20%. At a grouting depth of 30 m, as consolidation depth increases from 0.5 m to 4.0 m, for every 1 m increase in consolidation depth, the allowable grouting pressure increases by 2.0 MPa, and λ decreases by 0.06. Under comparable conditions at a depth of 5 m, for every 1 m increase in consolidation depth, the allowable grouting pressure increases by 2.0 MPa and λ decreases by 0.01. The increase in the allowable grouting pressure due to consolidation depth is not affected by grouting depth, but the variation

of λ is affected by grouting depth. This is manifested as: at a grouting depth of 30 m, as consolidation depth increases from 0.5 m to 4.0 m, λ decreases by 69.2%; at a depth of 5 m, the comparable decrease is 23.6%. The reason is that while the increase in the allowable grouting pressure due to consolidation depth is consistent, shallower grouting depths have smaller initial pressures, leading to this phenomenon.

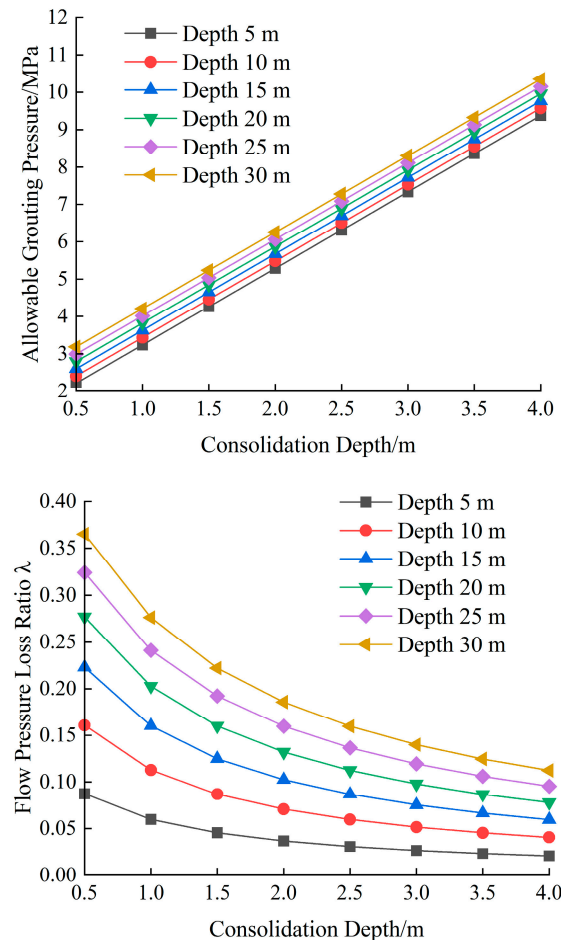


Figure 8. Variation curves of allowable grouting pressure and flow pressure loss ratio with consolidation depth.

According to the linear relationship between opening and fracture initiation pressure, the variation curves of the allowable grouting pressure and the flow pressure loss ratio with fracture initiation pressure can be obtained.

Analysis of Figure 9 reveals that the allowable grouting pressure increases significantly with the increase in fracture initiation pressure. For every 20% increase in fracture initiation pressure, the allowable grouting pressure increases by 25%–30%. At a grouting depth of 30 m, when the fracture initiation pressure increases from 0.5 MPa to 4.0 MPa, the allowable grouting pressure increases by 4.8 times. When the fracture initiation pressure increases from 3.0 MPa to 4.0 MPa, the pressure increase magnitude is 5.2 MPa, exhibiting strong nonlinear characteristics. The flow pressure loss ratio rapidly decreases with the increase in fracture initiation pressure. At a grouting depth of 30 m, λ decreases by 79.3%, while at a depth of 5 m, the comparable decrease is 10.0%. This indicates that as grouting depth increases, the decrease magnitude of λ significantly expands. This phenomenon reveals that higher fracture initiation pressure can more effectively reduce λ . In engineering applications, higher fracture initiation pressure should be adopted for deep grouting to

form a higher pressure barrier effect, support greater grouting pressure, and simultaneously achieve effective cyclic grouting.

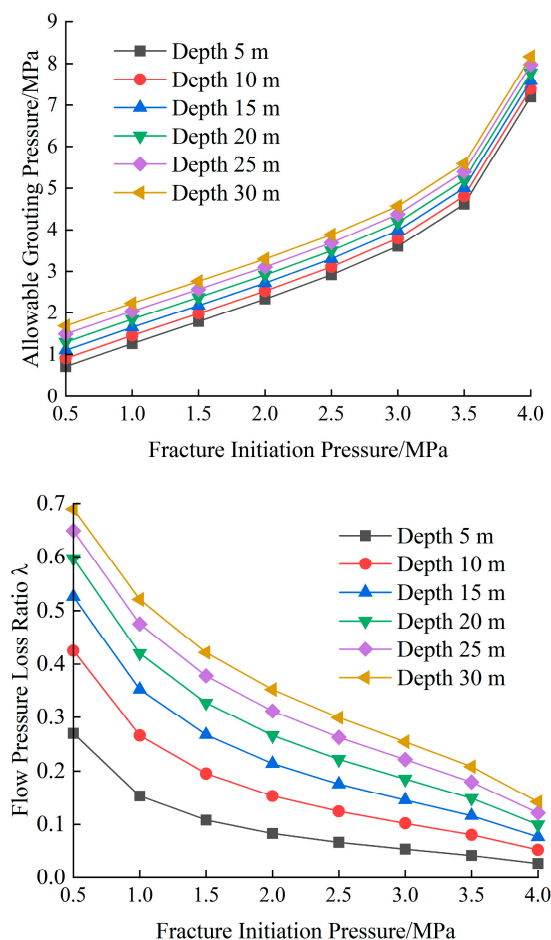


Figure 9. Variation curves of allowable grouting pressure and flow pressure loss ratio with fracture initiation pressure.

4. Experimental Materials and Methods

This chapter introduces the experiments and methods used to verify the high-pressure cyclic grouting technology. The experimental preparation work is conducted from the aspects of grouted medium, grouting materials, grouting process and working condition design, and post-grouting effect evaluation.

4.1. Soil Characteristics

A certain inland harbor basin in Fujian was constructed on a tidal flat area for the transportation of prefabricated beams at sea. The site has a mucky clay layer with a depth of 15 m. The harbor basin is 38.5 m in width and 130 m in length, with a site elevation of 5.0 m and a basin bottom elevation of -4.0 m, requiring an excavation depth of 9 m. Beam transportation platforms with a top elevation of 4.5 m were installed on both sides. Due to the high groundwater level and the threat of dynamic water erosion to the stability of the seaside slope, grouting treatment and reinforcement are proposed, and on-site grouting tests are currently being conducted.

To obtain the compression characteristic parameters of the mucky clay at the site, laboratory-confined compression tests were conducted in accordance with the GB/T 50123-2019 [23]. Core sampling was performed using a 91 mm diameter drill, and four groups of undisturbed soil cores were extracted from the depth range of 2.0 to 4.0 m. The soil cores

were immediately sealed after extraction to avoid disturbance. In the laboratory, specimens were prepared using cutting rings with an inner diameter of 61.8 mm and a height of 20 mm, with one standard specimen prepared from each soil core group. Single-sided drainage incremental loading tests were performed using a fixed-ring rigid compression apparatus, with staged pressures of 50 kPa, 100 kPa, 200 kPa, 400 kPa, 800 kPa, 1600 kPa, 3200 kPa, and 5000 kPa, with each stage maintained for 24 h. The hyperbolic equation was converted into a linear form, with P as the abscissa and P/ε as the ordinate. A scatter plot was drawn, and linear fitting was performed using the least squares method. The slope of the resulting line represents the m value, and the y -axis intercept represents the initial compression modulus E_0 . The fitting results for each specimen are shown in Table 1.

Table 1. Formation experimental parameters at different depth positions.

Depth Position (m)	Initial Compression Modulus (MPa)	Parameter (m)
2.0~2.5	3.4	3.4
2.5~3.0	3.5	3.1
3.0~3.5	3.6	3.2
3.5~4.0	3.5	3.1

According to Table 1, the average compression modulus is 3.5 MPa and the average parameter m is 3.2. The test results can provide support for subsequent theoretical calculations of the grouting reinforcement radius.

4.2. Grouting Material Characteristics

To obtain the rheological parameters of cement paste, tests were conducted using a Thermo Scientific Haake Mars 60 soft solid rotational rheometer (Thermo Fisher Scientific, Waltham, MA, USA) with a shear rate range of 0.01 s^{-1} to 1000 s^{-1} . The test procedures were performed in accordance with ASTM C1749-24 [24].

To ensure the repeatability of the tests, pre-shearing was performed at a rate of 200 s^{-1} for 60 s before the formal testing, followed by a 30 s rest period. A variable shear rate mode was adopted, with the shear rate linearly increasing from 2 s^{-1} to 50 s^{-1} , maintaining an equilibration time of 15 s at each rate gradient. The experimental results are shown in Figure 10.

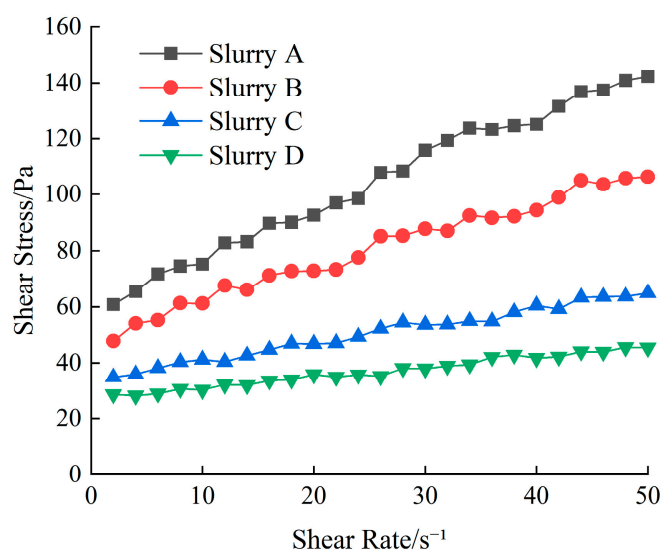


Figure 10. Rheological test results of cement paste with different mix proportions.

The Bingham model was applied to fit the data in Figure 10, yielding Table 2, which shows that the rheological equations of pastes with different mix proportions differ significantly, but all conform to Bingham fluid characteristics, and viscosity is positively correlated with yield stress.

Table 2. Fitted rheological equations of different slurries.

Mix Proportion	Fitted Equation	Yield Stress (Pa)	Plastic Viscosity (Pa·s)	Fitting Coefficient (R ²)
A	$\tau = 1.71\gamma + 60.28$	60.28	1.71	0.990
B	$\tau = 1.17\gamma + 50.16$	50.16	1.17	0.983
C	$\tau = 0.63\gamma + 34.32$	34.32	0.63	0.986
D	$\tau = 0.38\gamma + 27.10$	27.10	0.38	0.982

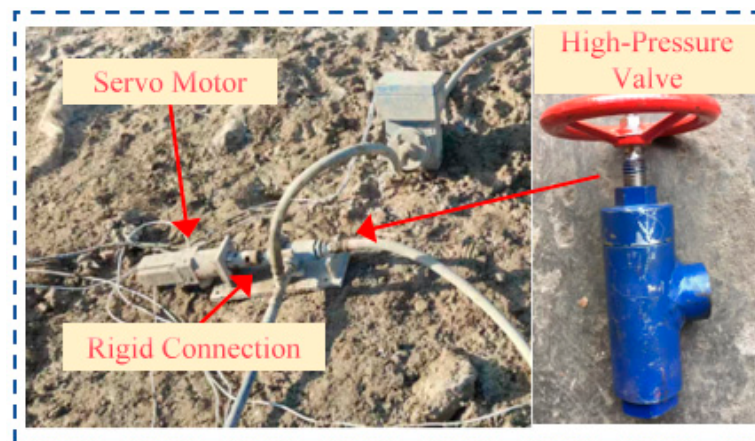
4.3. High-Pressure Cyclic Grouting System and Experimental Design

A system integrating flow and pressure monitors with high-pressure valves was adopted. Flow and pressure monitors were installed at the grout inlet and outlet, respectively. The return valve was rigidly connected to a servo motor, and a screw-type high-pressure valve was selected. The servo motor can achieve complete opening or closing of the valve by rotating 5.5 turns. A control program was developed using the PLC1200 module to collect and monitor flow and grouting pressure data at the inlet and outlet in real time. When the system determines that pressure adjustment is required, the servo motor precisely controls the opening and closing amplitude based on the current opening ratio of the valve. For example, when the injection rate is lower than 5 L/min, the system commands the return valve to close, reducing the flow cross-sectional area of the return channel, thereby increasing the grouting pressure and achieving the goal of improving the injection rate. The core performance indicators of the servo motor–valve linkage system are as follows: response time < 1 s, and switching time from fully closed to fully open < 5 s. The supporting equipment for this system includes a 3SNS-A three-cylinder grouting pump with a rated grouting pressure of 8 MPa and an injection rate of 100 L/min to meet on-site requirements. The structure of key parts is shown in Figure 11.

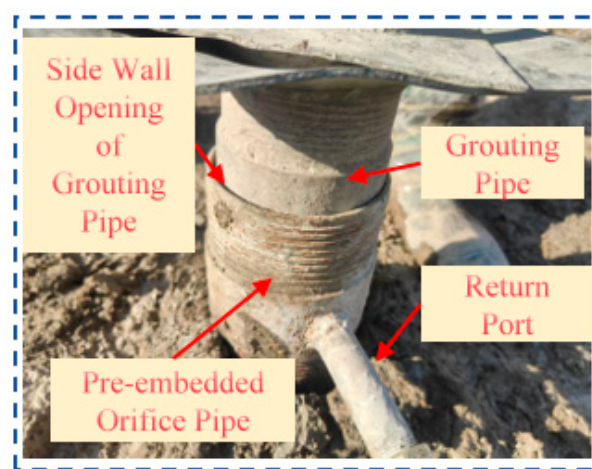
Two groups of grouting experiments were conducted. In the first group, surface pretreatment was performed in the area to a depth of 1.5 m, and the surface fracturing pressure P_f after treatment was measured to be 1.5 MPa. The embedded pipe had a radius of 6 cm and a depth of 1.5 m; the injection pipe had a radius of 5.5 cm, resulting in a side wall opening of 0.5 cm for the injection pipe, with an h_0 of 0.25 cm; the crack opening b at seal failure was 0.2 cm. The injection rate was 15 L/min, using Group A stable grout with a yield stress τ_0 of 60.28 Pa and a plastic viscosity μ of 1.71 Pa·s; the grouting section length L was 1 m. In the control group, the surface was not pretreated, the injection pipe radius was 5.5 cm, and the grouting depth, injection rate, and other parameters were kept consistent with the first group.

4.4. Post-Grouting Effect Evaluation and Detection Methods

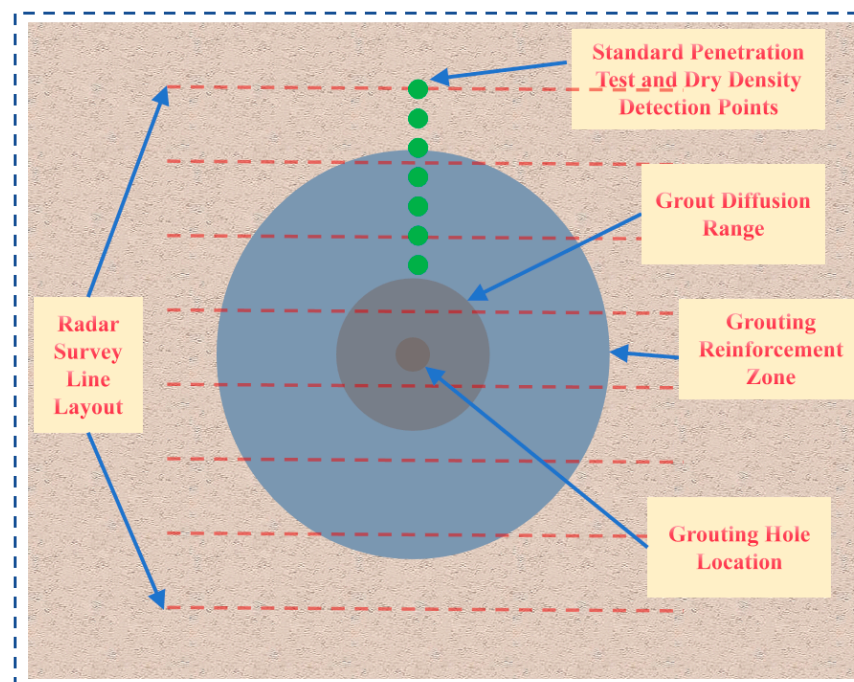
Non-destructive inspection was conducted with a 200 MHz GPR (RTS-200, Huace Technology Co., Ltd., Beijing, China). Eight horizontal survey lines were arranged, with one line every 40 cm, and the length was set to 1.5 times the calculated influence diameter. The layout of the survey lines is shown in Figure 12.



(a) Servo motor–valve linkage apparatus



(b) Orifice structure

Figure 11. Photographs of the high-pressure cyclic grouting system.**Figure 12.** Field experiment layout plan.

In situ standard penetration tests were conducted in the grouting reinforcement area. Test points were arranged at 20 cm intervals along the radar survey line direction, totaling 7 test points, with 91 mm boreholes drilled and simultaneous core sampling. According to the ASTM D1586/D1586M-18 [25], a 63.5 kg hammer was used to drive the split-spoon sampler with a fixed drop height of 76 cm. Before the test, a 15 cm pre-penetration was performed to eliminate the influence of drilling, and then the number of blows required for a further 30 cm penetration, i.e., the N-value, was recorded. The retrieved undisturbed core samples were cut into standard dimensions with a diameter of 5 cm and a height of 2 cm, dried and weighed, with 3 parallel samples prepared for each group to calculate the dry density of the core samples.

5. Results and Discussions

5.1. Analysis of Grouting Results

To investigate the deformation pattern of the injection pipe side wall opening and determine the range of the opening coefficient, cyclic grouting was performed before the experiment. After the grout solidified, the injection pipe was pulled out, and the thickness of residual grout on the pipe wall was observed. As shown in Figure 13, along the grout diffusion direction, the injection pipe side wall opening gradually decreased. Specifically, the maximum wall opening deformation of 4.0 cm occurred at the 4.5 m depth; the wall openings at 3.0 m and 2.5 m were 2.0 cm and 1.5 cm, respectively, all significantly larger than the initial opening of 0.5 cm at 1.5 m. The goodness of fit for the four sets of data reached 0.996, showing a good linear relationship and verifying the assumption of linear variation of opening with depth. The opening coefficient $n = 7.82$ was obtained from the ratio of the fitted opening of 3.91 cm at 4.5 m to the initial opening of 0.5 cm, and the reference value of n was taken as 7.8.

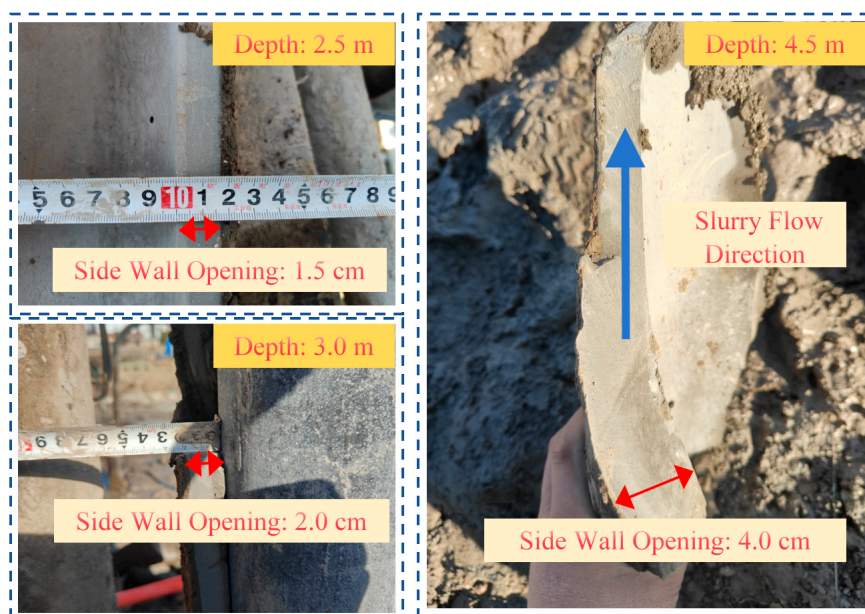


Figure 13. Variation of injection pipe side wall opening.

The grouting results are shown in Table 3. For grouting holes without surface treatment, grout overflow at the borehole collar occurred at a grouting pressure of 0.21 MPa when the hole depth was 3.5 m, resulting in low grout injection volume and difficulty in ensuring grouting effectiveness. In contrast, for the experimental holes using surface reinforcement, pre-embedded pipe sealing, and cyclic grouting, a grouting pressure of 4.2 MPa was achieved at a depth of 4.5 m, reaching 94% of the theoretical grouting pres-

sure value, without grout overflow at the borehole collar, indicating that this technique significantly increased the allowable grouting pressure at the borehole collar to 20 times the normal value.

Table 3. Grouting experimental data table.

Test No.	Grouting Depth (m)	Grouting Section Length (m)	Cumulative Injection Volume (m ³)	Grouting Pressure (MPa)	Theoretical Allowable Grouting Pressure (MPa)	Borehole Condition
1	1.5~2.5	1.0	0.68	3.8	4.41	No abnormality
	2.5~3.5	1.0	0.65	3.9	4.43	No abnormality
	3.5~4.5	1.0	0.72	4.2	4.45	No abnormality
2	1.5~2.5	1.0	0.046	0.13	-	Slurry overflow along pipe wall
	2.5~3.5	1.0	0.051	0.21	-	Slurry overflow along pipe wall

5.2. Evaluation of Grouting Effectiveness

The ground-penetrating radar detection results are shown in Figure 14a–c. Amplitude-based qualitative interpretation has been widely validated in grouting engineering [26–28]. In the grouting reinforcement zone from 1.5 m to 4.5 m, the radar wave reflection signals exhibited moderate amplitude overall, with good continuity of the reflection events. This indicates that after high-pressure cyclic grouting, the soil pores were effectively compressed and the density was significantly improved. The continuous reflection with moderate amplitude further indicates that the grout formed uniformly distributed consolidation masses under high pressure, which, together with the surrounding compacted soil, constituted a reinforced body. In the shallow pretreatment zone from 0 m to 1.5 m, the signals were mainly of high amplitude, which was due to the formation of a strong rigid reflection interface in the shallow soil after pretreatment, while the locally occurring low amplitude reflected the local differences in the dielectric constant of the shallow medium. In the zone below 4.5 m, the signal rapidly attenuated to low amplitude, indicating that the area below this depth was basically undisturbed soil with no signs of reinforcement. The reflection attenuation trends in the horizontal 2.0 m to 3.0 m interval were consistent across the three isosurface images, demonstrating the reliability and repeatability of the detection data.

The detection results of the standard penetration test and sampling density experiment are shown in Figure 15:

According to the detection data analysis, the formation reinforcement effect is significant: within 0.4 m from the grouting hole orifice, the soil dry density increases by 16%, and the standard penetration blow count increases to 24, far exceeding conventional compactness indicators, indicating that the slurry compaction effect is significant in this area. At the 1.0 m position, the core sample dry density is greater than 2.9 g/cm³, and the standard penetration blow count is greater than 12, with the improvement still obvious. In the area beyond 1.0 m, the density and standard penetration blow count decreased by 16% and 30%, respectively. Beyond 1.2 m, these parameters returned to the undisturbed soil level, indicating that the effective grouting reinforcement radius is approximately between 1.0 m and 1.2 m. From the depth perspective, the reinforcement effect at a depth of 3.0 m is optimal. At the 0.4 m position within this depth, the density reaches 3.3 g/cm³, and the standard penetration blow count is 26. Both the compactness and strength attenuation gradient are superior to those at depths of 2.0 m and 4.0 m, forming a stable composite reinforcement layer.

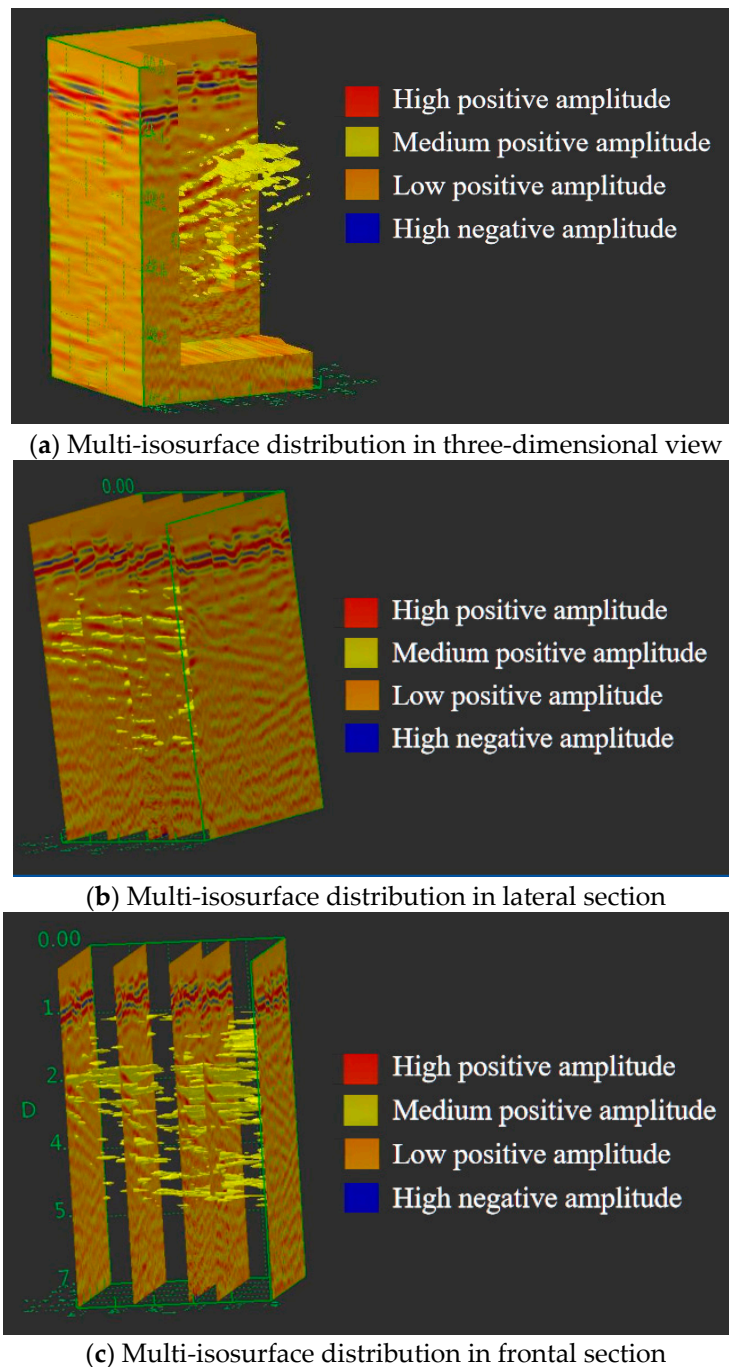


Figure 14. Ground-penetrating radar detection data.

5.3. Comparison of Theoretical and Experimental Results

As shown in Table 4, the reinforcement radius measured in the field test ranged from 1.0 m to 1.2 m, which agreed well with the theoretical predicted values ranging from 0.9 m to 1.0 m. The maximum error was maintained within 18%, and the average error was below 15%. The errors were mainly attributed to three factors: (1) the model assumes the soil to be a homogeneous medium, while the actual soil exhibits heterogeneity and anisotropy; (2) measurements are subject to limitations, such as errors from pressure sensors and ground-penetrating radar; and (3) although cyclic grouting can effectively avoid stress concentration, the soil may undergo a transition from compaction to micro-fracturing under the high pressure of 4.2 MPa, and the transient changes in pore water pressure are not fully coupled in the current simplified model. Although the complex factors noted above

are idealized in the model, an error level of 15% is completely acceptable in engineering practice. This indicates that the simplified model effectively captures the dominant physical mechanisms of high-pressure cyclic grouting, thereby verifying its rationality and reliability in engineering applications.

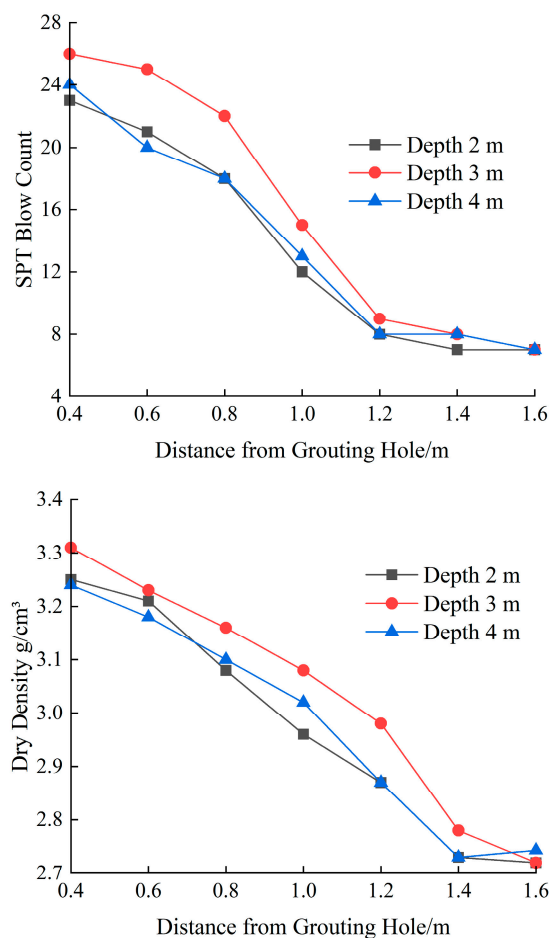


Figure 15. Blow count and dry density detection data diagram.

Table 4. Comparison of theoretical and actual reinforcement ranges.

Grouting Depth (m)	Theoretical Reinforcement Radius (m)	Measured Reinforcement Radius (m)	Relative Error (%)
1.5~2.5	0.94	1.0~1.2	14.55
2.5~3.5	0.91	1.0~1.2	17.27
3.5~4.5	0.96	1.0~1.2	12.73

5.4. Limitations and Future Research Directions

Although the proposed technology has been successfully verified in the field, some limitations remain due to simplified assumptions and the scope of this study. The following discusses these aspects and future research recommendations.

(1) Simplification of constitutive relations and fluid–solid coupling

This study adopts a hyperbolic model based on total stress that cannot reflect the pore water pressure dissipation process and thus limits the predictive capability for long-term consolidation settlement of soft clay. In the future, the Modified Cam-Clay model [29] should be introduced and coupled with Biot's [30] consolidation theory to more accurately

describe the evolution of effective stress paths and consolidation behavior of soil. In addition, the model assumes idealized fracture geometry, laminar flow conditions, and isotropic permeability. Sensitivity analysis is needed in the future to quantify the impact of these simplified assumptions on the prediction of reinforcement range.

(2) Long-term performance and durability

This study was verified for the short-term requirements of temporary cofferdam reinforcement. Considering the application potential of this technology in permanent projects such as dams, its long-term service performance, such as cyclic load fatigue and groundwater erosion, requires further research. Future research should combine accelerated aging tests to establish a corresponding durability evaluation system.

6. Conclusions

Through theoretical analysis and experimental verification, this paper proposes a surface consolidation and top-down cyclic grouting process, clarifies the influencing factors of the allowable grouting pressure, investigates the slurry grouting effect under specific grouting mechanisms, and verifies the correctness of the theory and its engineering applicability through field experiments. The main conclusions are as follows:

- (1) A surface consolidation and top-down cyclic grouting process is proposed. A grouting pressure loss equation is established to quantitatively analyze the relationships between the allowable grouting pressure and the side wall opening of the grouting pipe, surface consolidation depth, surface consolidation strength, and slurry rheological parameters. Based on the strain–pressure curve, a prediction model for the reinforcement radius of compaction grouting is established.
- (2) Compared with yield stress, viscosity variation has a more significant effect on the allowable grouting pressure. The increase in the side wall opening of the grouting pipe causes the allowable grouting pressure to exhibit two-stage attenuation. The influence of the opening coefficient on the allowable grouting pressure becomes increasingly pronounced with increasing burial depth. Ignorance of the variation of the opening can result in a deviation in the allowable grouting pressure ranging from 4.6% to 21.2%. For every 20% increase in the surface consolidation depth, the allowable grouting pressure increases by 15% to 20%, while for every 20% increase in the fracture initiation pressure, it increases by 25% to 30%. The flow pressure loss ratio decreases with an increase in viscosity, increases with an increase in yield stress, and decreases significantly with an increase in the opening. To ensure the effectiveness of cyclic grouting, slurries with high viscosity and low yield stress may be adopted for deep grouting while simultaneously increasing the side wall opening of the grouting pipe.
- (3) Formation compression modulus parameters and slurry rheological parameters were obtained through confined compression tests and rheological tests, and comparative tests of layered grouting were conducted. The tests show that after 1.5 m surface consolidation pretreatment, the grouting pressure at the 4.5 m depth can be increased to 4.2 MPa with no slurry leakage on the ground. Within a 1.0 m radius, the core sample dry density is greater than 2.9 g/cm³, an increase of more than 8%; the standard penetration blow count is greater than 12, an increase of more than 30%; and radar analysis indicates uniform grouting reinforcement. Comprehensive results show that this grouting process greatly improves the bearable grouting pressure in the near-surface area and effectively improves the uniformity of grouting reinforcement.

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