



Article Quantitative Design Method for Grouting in Sand Layers: Practice in Qingdao Metro Line 2

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Abstract: Grouting is an effective method to reduce permeability and improve the mechanical performance of sand layers, preventing a disastrous inrush of sand and water. A scientific grouting design scheme is the premise for satisfying grouting reinforcement requirements. Due to a lack of theoretical basis for current grouting designs, grouting projects are conducted empirically and blindly. This paper presents a quantitative design method for grouting in sand layers. Based on this method, a quantitative design is realized for judgment of the grouting mode, determination of grouting range and calculation of grouting reinforcement effect. Moreover, for the fracture-compaction grouting mode, a theoretical model is proposed to calculate the grouting process, considering the coupling effect of grout flow and sand layer deformation. Meanwhile, a calculation method for reinforcement is put forward, which can connect macroscopic performance of the grouted body and individual performance of grout veins, compacted sand and undisturbed sand. In order to verify the efficiency of the grouting design method, it has been used in a sand grouting project in Qingdao Metro Line 2. In this project, judgment of the grouting mode, selection of grouting type and determination of grouting parameters have been completed based on the design method. Several inspection approaches have been performed to evaluate the effectiveness of the grouting design, showing that engineering stability was guaranteed after the grouting operation.

Keywords: sand; grouting design; permeation mode; fracture-compaction mode; diffusion; reinforcement effect

1. Introduction

Sand layers with high saturation, low cementation degree and poor self-stability are often exposed in tunneling excavations. Affected by excavation disturbance and seepage flow, engineering accidents usually occur when excavating tunnels in sand layers, such as the collapse of the tunnel and inrush of sand and water. Grouting is an effective method for treatment of unfavorable strata, such as loose layers and sand layers [1–3]. By injecting the cemented grout material into the weak or damaged ground through the constant grout take or pressure device, the grout slurry diffuses, gels, and solidifies (curing time approximately 24 h) in the ground to reduce the under-groundwater inflow, improve the mechanical properties and achieve reinforcement of weak regions. In several treatment projects of unfavorable strata, grouting methods have played an important role and their effectiveness has been verified [4–7]. However, the grain structure of sand layers is complex and changeable, and the use of grouting modes in sand layers is uncertain. It is hard to



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Copyright: © 2022 by the authors. Licensee MDPI, Basel, Switzerland. This article is an open access article distributed under the terms and conditions of the Creative Commons Attribution (CC BY) license (https:// creativecommons.org/licenses/by/ 4.0/). acquire satisfying performance after grouting operations in sand layers. Consequently, grouting in sand layers is technically more challenging than in other strata.

Many sand grouting projects indicate that grouting modes in sand layers can be divided into a permeation mode and a fracture-compaction mode. Grouting mode in sand layers mainly depends on sand groutability. A series of tests have been performed to study groutability of sand layers. Based on comparison between characteristic grain sizes of grout and injected sand, several formulas predicting sand groutability have been developed [8–11]. The permeation grouting mode has been widely studied considering many influencing factors, which mainly involve time-dependent behavior of grout viscosity [12–16], filtration effects [17–21] and rheological characteristics of grout [18,22,23]. Furthermore, many calculation models and numerical methods to acquire grouting radius for permeation grouting have been established. In aspects of the fracture-compaction grouting mode in sand, Bezuijen [3] reported on a number of experiments and field experiences for grouting in sand layers, and connections among fracture shape, grouting parameters and density of the sand were described qualitatively. Wang et al. [24] carried out laboratory-scale tests on loose sand under confined boundary conditions, and influences of water/cement ratios and saturation degree on characteristics of the grouted bulbs (dimension and shape) were obtained. Zhang et al. [25] developed an analytical model to describe the grouting process of the fracture–compaction mode considering coupling effects of grout flow and deformation of the grouted medium. However, there is little research on analysis methods involving how to acquire macroscopic performance of the grouted body. As mentioned above, there are abundant research results about grouting mechanisms in sand layers. However, few research results have been effectively applied in engineering projects.

Grouting design method is the bridge connecting basic grouting theory and engineering applications. An effective and implementable grouting design scheme for sand layers devised before the grouting operation is the premise for satisfying grouting reinforcement requirements of sand layers. Current grouting designs are mainly based on engineering experiences and similar projects. Grouting pressure is usually set at several times (1.5~3 times) hydrostatic pressure. Grout take is usually determined by the porosity of the target region of grouting. The most optimal grouting parameters can hardly be determined according to the actual situation of projects. Furthermore, due to a lack of prediction of the macroscopic performance of the grouted body, engineering stability and outflow of water cannot be analyzed effectively. In general, current grouting projects are conducted empirically and blindly.

With the objective to realize quantification of grouting designs in sand layers, this paper presents a systemic grouting design method and corresponding design procedure. The novel feature of this method is that quantitative design is simultaneously realized in judgment of grouting mode, determination of grouting range and calculation of the grouting reinforcement effect. This method has been used successfully in the sand grouting project of Beer-miao running tunnel, a part of Qingdao metro line 2, in China.

2. Quantitative Design Method for Sand Grouting

2.1. Design Procedure

Design procedure for sand grouting is shown in Figure 1, including seven steps detailed below.



Figure 1. Design procedure for sand grouting.

(1) Determination of grouting design objective

The design objective of grouting in sand layers mainly involves engineering stability, deformation of the surrounding rock and outflow of water. Engineering stability is the most basic objective. Deformation of surrounding rock and outflow of water should be determined by related engineering codes, considering the type and importance of engineering projects.

(2) Determination of characteristic indexes of sand

There are several characteristic indexes of sand, which are related to grouting diffusion process and reinforcement effect [7,11,25]. The characteristic indexes of sand mainly involve

grain size distribution, cohesive soil content, relative density and in situ stress. Grain size distribution, cohesive soil content and relative density of the sand layer can be measured by laboratory testing. In situ stress can be measured by in situ tests or estimated by the buried depth of the projects.

(3) Judgment of grouting mode

Many sand grouting projects indicate that grouting modes in sand can be divided into a permeation mode and a fracture–compaction mode. For the permeation mode, grout is injected into pores between sand grains, with the sand grain skeleton not being destroyed. After this, a relatively homogeneous grouted body will form. For the fracture– compaction mode, the sand layer is fractured by injection pressure and is compressed on both sides of the fracture. Eventually, a markedly anisotropic grouted body will form, which is composed of the grout vein, compacted sand and undisturbed sand. There are significant differences between these two grouting modes in aspects of the grouting diffusion process and reinforcement effect. Therefore, the grouting mode should be judged prior to calculation of the grouting range and effects.

(4) Selection of calculation method of grouting diffusion process

The purpose of calculation of the grouting diffusion process is to acquire a quantitative relationship between grouting parameters (injection pressure, injection rate, grout take), rheological properties of grout and diffusion results (grouting radius, shape of grouted body, et al.), which is the basics for grouting design. For the permeation mode, there are significant differences in diffusion characteristics for different grouts. Regarding quick-setting grout, such as cement–sodium silicate grout, time-dependent viscosity behavior has a significant effect on the grout diffusion process. Regarding suspension grout, such as cement grout, there exists filtration during injection. For the fracture–compaction mode, the coupling effect of grout flow and sand layer deformation has control over grout diffusion. When selecting a calculation method of the grouting diffusion process, grouting mode and grout characteristics should be considered adequately.

(5) Selection of calculation method of grouting reinforcement effect

The purpose of calculation of grouting reinforcement effect is to determine the quantitative relationship between grouting parameters (injection pressure, injection rate, grout take), grout characteristics and performance of the grouted body. Once performance parameters of the grouted body are determined, engineering stability, deformation of surrounding rock and outflow of water can be calculated subsequently. For the permeation mode, the macroscopic performance of the grouted body can be represented by performance of local elements, which can be measured by laboratory tests. For the fracture–compaction mode, the grouted body is composed of three different regions, including the grout vein, compacted sand and undisturbed sand. With the aim to understand the macroscopic performance of the grouted body, firstly, performance parameters of three different regions should be measured by laboratory tests. Secondly, adequate calculation methods should be selected to connect the macroscopic performance of the grouted body and performance of three different regions, considering the spatial relations among the three different regions.

(6) Preliminary grouting design

Firstly, according to similar sand grouting projects and engineering experiences, specific design content should be determined preliminarily, such as grout (type and mix ratio), grouting parameters (injection rate, injection pressure and grout take), grouting reinforcement scope and arrangement of injection holes. Secondly, calculation of grouting diffusion process and reinforcement effect will be carried out to acquire grouting range and reinforcement effect. Thirdly, based on the calculation results above, engineering stability, deformation of surrounding rock and outflow of water will be analyzed to determine whether the design is effective. Finally, grouting design content will be optimized according to the engineering objective and economic requirements.

(7) Grouting design adjustment

After implementation of the grouting design, several inspection approaches will be performed to evaluate the effectiveness of the grouting design. Subsequently, the grouting design will be adjusted according to the evaluation results. Inspection approaches include dynamic monitoring data analysis (p-q-t), outflow of water in checkhole, borehole television, observation of the grouted body exposed in the tunnel face and monitoring of tunnel deformation.

2.2. Judgment of Grouting Mode

The chosen grouting mode in sand mainly depends on sand groutability. When groutability is successful, the permeation grouting mode will be used (Figure 2a). The fracture–compaction grouting mode corresponds to unsuccessful groutability (Figure 2b). Several grouting tests have been carried out to study sand groutability, and several formulas to predict groutability were developed [8–11]. The most important parameters in these formulas are the characteristic grain size of grout and injected sand. Four representative groutability criteria of sand are shown in Table 1. In engineering projects, several groutability criteria should be used simultaneously to evaluate sand groutability and judge grouting mode. Successful and unsuccessful groutability correspond to the permeation mode and fracture–compaction mode, respectively. When groutability is insufficient, further field tests should be carried out to determine grouting mode.



Figure 2. Grouting mode in sand layer. (a) Permeation mode. (b) Fracture-compaction mode.

Literature	Formulas for Predicting Groutability	Groutability
Burwell (1958)	$N = D_{15}/d_{85}, \ M = D_{10}/d_{95}$	N > 25, M > 11 corresponds to successful; $N < 11, M < 5$ corresponds to unsuccessful
Mitchell (2006)	$N = D_{15}/d_{85}, \ M = D_{10}/d_{95}$	N > 24, M > 11 corresponds to successful; $N < 11, M < 6$ corresponds to unsuccessful
Akbulut and Saglamer (2002)	$N = \frac{D_{10}}{d_{90}} + K_1 \frac{W/C}{FC} + K_2 \frac{P}{Dr}$	N > 28 corresponds to successful. N < 28 corresponds to unsuccessful
Zhang (2017)	$N = \frac{(1 - 0.2D_r)(1 - 1.1\theta)D_{15}}{(1.2 - 0.2W/C)d_{85}}$	N > 31 corresponds to successful; $25 \le N \le 31$ corresponds to insufficient; N < 25 corresponds to unsuccessful

Table 1. Groutability criteria of sand.

Where *N*, *M* is groutability of sand; D_{10} and D_{15} are particle diameters at which 10% and 15% of the weight of sand is finer, respectively; d_{85} , d_{90} , and d_{95} are particle diameters at which 85%, 90% and 95% of the weight of grout is finer, respectively; *W*/*C* is water/cement ratio of grout; *FC* is finer content of sand passing through 0.6 mm sieve; *P* is grouting pressure; D_r is relative density of sand; K_1 , K_2 is constant based on test experiences; θ is cohesive soil content.

2.3. Calculation of Grouting Range and Effects for Permeation Mode

(1) Calculation model considering time-dependent behavior of grout viscosity

Time-dependent behavior of grout viscosity was tested by several laboratory tests [12–16], and several equations were established which describe the relationship between grout viscosity and grout reaction time. The permeation grouting process considering time-dependent behavior of grout viscosity was widely studied, and several analytical models and numerical simulation methods were developed [18,21–23]. Based on the research results above, for quick-setting grout, the quantitative relationship between grouting parameters (injection pressure, injection rate), rheological properties of grout and diffusion range can be acquired.

(2) Calculation model considering filtration

For suspension grout (such as cement-based grout), if there is no significant difference between pore size of the injected medium and grain size of grouts, filtration controls action on grouting process. Maghou et al. [17] studied filtration by laboratory tests and developed a linear filtration law which describes the connections among filter velocity of grout, pore size of sand skeleton and concentration of grout. Based on the linear filtration law, several calculation methods for different diffusion types, such as 1-dementional type, cylindrical type and spherical type, were developed [18–21]. Based on existing calculation methods, the distribution of porosity of the injected medium and retention rate of grout under different grouting parameters can be determined. Eventually, effective grouting range can be determined.

(3) Determination of permeation grouting reinforcement effects

For the permeation mode, performance parameters of local elements of the grouted body, under different mix ratios of grout and ages of grouted body, can be acquired by laboratory tests. To guarantee accordance with in-situ strata in aspects of grain size distribution and component contents, grouted media in laboratory tests should be sampled from the engineering site. For grouting design of the permeation mode, grouting areas of adjacent injection holes overlap each other partially. Therefore, the grouted body can be considered approximately homogeneous for easy determination. Macroscopic performance of the grouted body can be represented by performance of local elements measured by laboratory tests.

2.4. Calculation of Grouting Range for Fracture-Compaction Mode

In terms of the fracture–compaction mode, as shown in Figure 3, grout is injected into the sand layer and fractures along the direction parallel to major principal stress. The shape of the grouted body can be seen as a 'round cake' which is perpendicular to minor major stress. The thickness of the grout vein decreases from injection hole to diffusion front.

The grouting diffusion process of the fracture–compaction mode is a coupling effect of the flow field of grout and the deformation field of the sand layer. In the grouting diffusion process, grout flow is subject to resistance from the fracture wall and grout pressure decreases along the fracture direction consequently. Due to the positive correlation between fracture width and grout pressure, fracture width decreases along the fracture direction as well. In other words, fracture width is determined by the grout pressure field in the fracture. Meanwhile, flow resistance from the fracture wall is determined by fracture width because of cubic law, which describes the law of fluid flow in fissure. The decay of fracture width along the fracture direction leads to differences in flow resistance at different position in the grouting area, and grout flow field will be changed subsequently. In other words, grout flow field in the fracture is affected by deformation of sand layer on both sides of fracture.



Figure 3. Sketch of fracture-compaction grouting process.

Based on the analysis above, calculation of fracture-compaction grouting diffusion involves a process of grout flow and sand compaction. The compaction process of sand is determined by the compressive force applied by grout, which is equal to grout pressure, and the compaction nature of the sand layer. The compaction nature of the sand layer can be characterized by the relationship between strain ε and pressure p. This relationship can be described by equation $\varepsilon = f(p)$ and measured by a confined compression test. In a real fracture-compaction grouting diffusion process, the grouting radius can reach tens of meters and fracture width can reach merely a few centimeters. There is a difference of several orders of magnitude between the grouting radius and fracture width. Therefore, the direction of sand compaction can be considered perpendicular to the fracture direction. In addition, the compaction process of the sand layer can be considered according with the lateral confinement condition approximately. As shown in Figure 4, within the influence range of grouting, there are grout zones in the fracture and compacted zones on both sides of the fracture. Without the influence range of grouting, there is an undisturbed zone which is not compacted in the grouting process. Fracture width is equal to the compaction deformation amount of the sand layer on both sides of the fracture and can be expressed as:

$$b = (f(p) - f(p_0))D$$
 (1)

where *b* is fracture width, *p* is grout pressure, p_0 is initial ground stress, f(p) is strain induced by p, $f(p_0)$ is initial strain in grouting area and *D* is influence range of grouting. Equation (1) describes the relationship between the grout pressure and fracture width. It indicates that the fracture width is controlled by the joint effect between grouting pressure, ground pressure, and grouting influence range.

Grout flow in fracture can be described approximately by fluid flow theory in fissure. Considering the rheological type of grout Bingham fluid, as analyzed in Figure 4, the governing equation of grout flow can be expressed by Equation (2) [10].

$$\frac{\mathrm{d}p}{\mathrm{d}r} = -\frac{12\mu\overline{v}}{b^2} - \frac{3\tau_0}{b} \tag{2}$$

where dp/dr is pressure gradient, μ is viscosity of grout, τ_0 is yield stress of grout, b is fracture width and \overline{v} is mean velocity in a cross section. Equation (2) describes the pressure gradient of a grout element in the fracture. The size of sand grains is not considered in this equation. The pressure gradient is related to fracture width, grout rheological parameter, and velocity.



Figure 4. Force analysis of grout flow.

Equation (1) is connected with Equation (2) by pressure continuity on the fracture wall. In other words, at the same position, the grout pressure in the fracture is equal to the compressive pressure applied on the sand layer. The coupling effect of grout flow and sand compaction deformation can be completely described by Equations (1) and (2). Combined with the initial conditions and boundary conditions, the fracture–compaction grouting diffusion process can be calculated. The solution of Equations (1) and (2) can be realized using the numerical method. Eventually, quantitative connections among injection rate, grouting pressure, rheological properties of grout, grouting radius and distribution of grout vein thickness (namely fracture width) can be acquired.

2.5. Calculation of Grouting Effects for Fracture-Compaction Mode

For the fracture–compaction grouting mode in sand layers, the grouted body is composed of three different regions, grout vein, compacted sand and undisturbed sand. As shown in Figure 5, several sand grouting projects of Qingdao metro line indicate that grout veins are approximately parallel with each other. Consequently, the performance of the grouted body is anisotropic. Performance of the grouted body in a direction parallel to grout veins is different from that in a direction perpendicular to grout veins. Besides, due to decay of the grout vein thickness from the injection hole to diffusion front, the grouting effect is inhomogeneous and decreases from the injection hole to diffusion front.

Performance parameters of three regions in the grouted body can be measured by laboratory tests. The difficulty is how to connect the macroscopic performance of grouted body and the individual performance of three different regions. Based on the observation of parallel distributed grout veins, a new simplified method is put forward to calculate the macroscopic performance of the grouted body for the fracture–compaction mode. The simplified calculation model is shown as Figure 6. This model satisfies three basic assumptions as follows:

- (1) Grout veins are parallel to each other;
- (2) Influence range of grouting is constant in the fracture direction;
- (3) For calculation convenience, injection holes are distributed at equal intervals and lines connecting injection holes are perpendicular to the fracture direction.



Figure 5. Typical grout veins in sand grouting projects of Qingdao metro line. (**a**) Beer-miao running tunnel in Qingdao metro line 2. (**b**) Shilaoren station in Qingdao metro line 2. (**c**) Taishan station in Qingdao metro line 4. (**d**) Jia-ling running tunnel in Qingdao metro line 13.



Figure 6. Simplified calculation model of grouting effect.

As shown in Figure 7, the representative element volume (REV) is selected from the above model which is symmetrical about the center of the grout vein. Upper and lower boundaries of the REV are located in the center of the undisturbed sand region, meaning that the height of the REV is equal to the interval of the injection holes *L*. Dimensions of the REV in the radial and circumferential direction are infinitely small, expressed by *dr* and *r*d θ respectively, in which *r* is the distance from the injection hole to the REV and θ is the circumferential angle of the REV to the injection hole. The area of the upper and lower end faces of REV is d*S*, which accords with the relation d*S* = *r*d*r*d θ . The influence range of grouting is expressed by *D*. As shown in Figure 7, when *L* is larger than *D*, the REV is composed of three regions, namely grout vein, compacted sand and undisturbed



sand. When *L* is smaller than or equal to *D*, there exists no region of undisturbed sand and the REV is composed of two regions, namely grout vein and compacted sand.

Figure 7. Representative element volume. (a) L > D. (b) $L \le D$.

The thickness of the grout vein is considered to be constant in REV, which is expressed by *b*. Thicknesses of compacted sand and undisturbed sand are expressed by h_1 and h_2 respectively. Geometric connections among the influence range of grouting, intervals of the injection holes and thicknesses of the three regions can be expressed as Equation (3).

D 1

$$\begin{cases}
h_1 = \frac{D-b}{2}, \ L > D \\
h_2 = \frac{L-D}{2}, \ L > D \\
h_1 = \frac{L-b}{2}, \ L \le D \\
h_2 = 0, \ L \le D
\end{cases}$$
(3)

where *L* is interval of injection holes, *D* is influence range of grouting, *b* is thickness of grout vein, h_1 is thickness of compacted sand and h_2 is thickness of undisturbed sand. Equation (3) provides a basic relationship regarding the thickness of undisturbed and compacted sand with grouting range. The thickness of undisturbed and compacted sand can be determined by the influence range of grouting (*D*), the interval of injection holes (*L*), and the thickness of the grout vein.

In this paper, anti-deformation performance and impermeability performance are characterized by modulus of compression E_s and permeability coefficient k, respectively. Anti-destruction performance is characterized by cohesion c and internal friction angle φ . Force and deformation analysis of the REV in the direction parallel and perpendicular to grout veins has been performed. The relationship between the macroscopic performance of grouted body and individual performance of three different regions have been acquired, which are expressed by Equations (4)–(7).

Macroscopic performance parameters of the grouted body can be determined using Equations (4)–(7). In the equations above, the thickness of the grout vein decreases from the injection hole to diffusion front. Distribution of thickness of the grout vein in the grouting area can be acquired based on the calculation results of the grouting diffusion process.

In real grouting design, injection holes are usually inhomogeneous and disperse due to restriction of working conditions. Therefore, the interval of injection holes is not

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constant. For convenient application of Equations (4)–(7), interval of injection holes *L* can be simplified to the average interval of all injection holes in target region of grouting.

$$\begin{cases} E_{sv} = \frac{L}{\frac{b}{E_{sb}} + \frac{D-b}{E_{s1}} + \frac{L-D}{E_{s2}}}, L > D\\ E_{sh} = E_{sb}\frac{L}{L} + E_{s1}\frac{D-b}{L} + E_{s2}\frac{L-D}{L}, L > D\\ E_{sv} = \frac{L}{\frac{L}{E_{sb}} + \frac{L-b}{E_{s1}}}, L \le D\\ E_{sh} = E_{sb}\frac{b}{L} + E_{s1}\frac{L-b}{L}, L \le D \end{cases}$$
(4)

$$\begin{cases} c_{v} = c_{b} \frac{b}{L} + c_{1} \frac{D-b}{L} + c_{2} \frac{L-D}{L}, L > D \\ \phi_{v} = \phi_{b} \frac{b}{L} + \phi_{1} \frac{D-b}{L} + \phi_{2} \frac{L-D}{L}, L > D \\ c_{v} = c_{b} \frac{b}{L} + c_{1} \frac{L-b}{L}, L \le D \\ \phi_{v} = \phi_{b} \frac{b}{L} + \phi_{1} \frac{L-b}{L}, L \le D \end{cases}$$
(5)

$$\begin{cases}
c_h = c_2, L > D \\
\varphi_h = \varphi_2, L > D \\
c_h = c_1, L \le D \\
\varphi_h = \varphi_1, L \le D
\end{cases}$$
(6)

$$\begin{cases} k_{v} = \frac{L}{\frac{k_{b}}{k_{b}} + \frac{D-b}{k_{1}} + \frac{L-D}{k_{2}}}, L > D\\ k_{h} = k_{b}\frac{b}{L} + k_{1}\frac{D-b}{L} + k_{2}\frac{L-D}{L}, L > D\\ k_{v} = \frac{L}{\frac{b}{k_{b}} + \frac{L-b}{k_{1}}}, L \le D\\ k_{h} = k_{b}\frac{b}{L} + k_{1}\frac{L-b}{L}, L \le D \end{cases}$$
(7)

where Φ_h , Φ_v are macroscopic performance parameters of the grouted body in the direction parallel and perpendicular to the grout veins respectively, Φ_b , Φ_1 , Φ_2 are performance parameters of grout vein, compacted sand and undisturbed sand respectively. E_s is modulus of compression, *c* is cohesion, φ is internal friction angle, *k* is permeability coefficient, Φ represents E_s , *c*, φ , *k*. Equations (4)–(7) present the relationship between the macroscopic performance parameter of the grouted body, the performance parameter of grout vein, compacted sand, and undisturbed sand. The macroscopic performance parameter of grouted body can be expressed by the performance parameter of grout vein, compacted sand, and undisturbed sand with their corresponding range.

3. General Situation of Project

3.1. Geological Condition and Support Design

This section presents a project involving grouting in a sand layer in which the quantitative design method developed in this paper has been used. This project is Beer-miao running tunnel, a part of Qingdao metro line 2. This tunnel is located in Qingdao city of China, down-traverse Hongkong Road, which is the busiest urban road in Qingdao. Beer-miao running tunnel is composed of two single line tubes, with tube spacing between 15 m and 17 m. The left tube has a length of 720.2 m, and the right tube has a length of 745.5 m. The buried depth of this tunnel is about 12~18 m.

In the influential area for the tunnel, the groundwater table is 1~4 m deep and strata mainly involve medium-coarse sand, clayey coarse-gravelly sand, intensely weathered granite and moderately weathered granite. Clayey coarse-gravelly sand with a thickness of 0.5~2 m is exposed in the upper semi-section of the tunnel face in the range of profile number 39 + 253~39 + 413 m, as shown in Figure 8. In the lower semi-section of the tunnel face, intensely and moderately weathered granite is exposed. The compressive strength of strata in the upper and lower semi-section of the tunnel face is listed in Table 2. The strata in the lower semi-section of the tunnel face have well stability in excavation process. However, there exists a huge risk when excavating the upper semi-section of the tunnel face. Clayey coarse-gravelly sand exposed in the roof arch of tunnel has almost no self-stability due to being water-saturated and having a loose structure. Furthermore, the tunnel is

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subjected to a heavy dynamic load due to large vehicle flow in the overlying urban road. Therefore, there exists a huge risk of inrush of sand and water when affected by excavation disturbance.



Figure 8. Geological longitudinal profile of the tunnel.

Table 2. Strength parameters of stratum in upper and lower semi-section of tunnel face.

Туре	Elastic Modulus/MPa	Poisson's Ratio	Cohesion/kPa	Internal Friction Angle/ $^{\circ}$
Upper semi-section	6.58	0.4	14.7	32.3
Lower semi-section	1300	0.325	600	33

Drill and blast technology is used in the construction of Beer-miao running tunnel according to the engineering design document from the officials. The excavation section of the tunnel has a width of 7.0 m and height of 7.43 m. Grid steel frames, shotcrete and bolt are used as primary supports. A small pipe grouting method is used as advance support. Reinforced concrete is used as secondary support. Detailed parameters of supports are shown in Table 3.

Table 3. Support design in tunnel.

Content of Support	Parameters		
Grid steel frame and shotcrete	Longitudinal space of 0.75 m, C25 early strength concrete, thickness of 30 cm		
Small pipe grouting	Scope of 120° on roof arch, diameter of 42 mm, length of 3.5 m, elevation of 15°, circumferential space of 0.3 m, longitudinal space of 1.5 m		
Bolt	Hollow bolt, diameter of 25 mm, length of 3.0 m, space of 1.0 \times 0.5 m		
Reinforced concrete	C45 reinforced concrete, thickness of 30 cm		

3.2. Process of Inrush of Sand and Water

When the tunnel excavation exposed clayey coarse-gravelly sand in the roof arch, small fall-block and seepage started to appear. Subsequently, an inrush accident of sand and water happened at profile number 39 + 253. The process of inrush of sand and water is shown in Figure 9a–c. After formation of the free face, a slight inrush occurred in the roof arch. A few minutes later, a massive inrush occurred with a mixture of sand and water outpouring into the tunnel. The mixture of sand and water is shown in Figure 9d and the

saturated sand has absolutely no self-stability. More and more strata of sand were lost in the inrush process. Eventually, there existed a subsidence of the ground surface due to stratum loss, as shown in Figure 10e.





Figure 9. Process of inrush of sand and water. (a) Excavation instant. (b) Early stage. (c) Late stage. (d) Mixture of sand and water. (e) Ground surface subsidence.



Figure 10. Grain size distribution of sand.

4. Grouting Design for Sand Layer

A grouting method was used in this project to reinforce the saturated clayey coarsegravelly sand.

4.1. Determination of Grouting Design Objective

The primary objective of this project is to guarantee engineering safety of the tunnel construction, and to avoid the occurrence of inrush accidents of sand and water.

- 4.2. Determination of Characteristic Indexes of Sand
- (1) Grain size distribution

Based on test results, the sand layer exposed in tunnel has a water content of ω = 20.2% and a cohesive soil content of θ = 14.91%. The grain size distribution of sand layer is shown in Figure 10. Control grain sizes of the sand layer are as follows: D_{10} = 0.043 mm, D_{15} = 0.08 mm, D_{30} = 0.45 mm, D_{60} = 1.75 mm. The finer content of sand passing through 0.6 mm sieve is 26.48%. The nonuniformity coefficient and curvature coefficient of the sand are C_u = 40.7, and C_c = 2.69, respectively.

(2) Compaction nature of sand

The compaction nature of the sand layer was measured by the confined compression test. The relationship between strain ε and pressure p was acquired and shown in Figure 11. A parabolic formula, Equation (8), is acquired to fit the test data. As shown in Figure 11, Equation (8) has good fitting accuracy. Equation (8) will be used in calculation of the grouting process of the sand.

$$\varepsilon = f(p) = 0.093\sqrt{p + 0.06} - 0.023 \tag{8}$$

where *p* is pressure in MPa, applicable range of formula: $p = 0 \sim 2$ MPa. Equation (8) is an empirical equation that describes the correlation between the strain and pressure. The empirical parameter can be obtained by fitting. The good agreement between the data and fitting curve as shown in Figure 11 demonstrates the accuracy of the empirical equation.



Figure 11. Relationship between strain *ε* and pressure *p*.

(3) Performance parameters of compacted sand

The modulus of compression of sand at different compaction pressures can be conveniently acquired using Equation (8). The cohesion, internal friction angle and coefficient of the sand at different void ratios was measured by laboratory tests. The relationship between the void ratio and strain of sand can be easily determined by soil mechanics theory. Meanwhile, the relationship between strain of sand and pressure can be acquired by Equation (8). Therefore, performance parameters of the sand at different compaction pressures would be determined, as shown in Figure 12. These performance parameters will be used in the calculation of the grouting reinforcement effect of the sand.



Figure 12. Relationship between performance parameters of sand and compaction pressure. (**a**) Modulus of compression and permeability coefficient. (**b**) Cohesion and internal friction angle.

(4) Initial state of sand layer

As mentioned above, the average depth of Beer-miao running tunnel is H = 15 m. The average unit weight of overlying strata is $\gamma = 20.4$ kN/m³. Using the average depth and unit weight of overlying strata, initial ground stress is estimated at $\sigma_0 = 306$ kPa. Based on Equation (8) and Figure 13, initial strain and performance parameters of sand layer can be acquired, as shown in Table 4.



Figure 13. Grain size distribution of cement.

Table 4. Initial strain and performance parameters of sand layer.

Initial Ground Stress $\sigma_0/{ m kPa}$	Initial Strain ε ₀	Initial Modulus of Compression E_{s0} /MPa	Initial Cohesion c ₀ /kPa	Initial Internal Friction Angle ${\varphi_0}^{/\circ}$	Initial Permeability Coefficient k ₀ /cm·s ⁻¹
306	0.033	14.09	14.7	32.29	$4.87 imes10^{-3}$

4.3. Judgment of Grouting Mode

As mentioned above, the grouting mode in sand mainly depends on sand groutability. Cement grout is used to estimate the groutability of the sand layer in this project. The water cement ratio interval of cement grout is $0.8 \sim 1.6$. The grain size distribution of cement used in this project is shown in Figure 13, and control grain size of grout are as follows: $d_{85} = 22.865 \mu m$, $d_{90} = 27.726 \mu m$, $d_{95} = 37.707 \mu m$.

It is difficult to measure the relative density of the sand layer before the inrush accident. Therefore, a medium dense degree of sand layer, $D_r = 50\%$, is used in the calculation of groutability. Based on the formulas for predicting groutability in Table 1, the calculation

results of groutability are shown in Table 5. For different water cement ratios and different formulas, all groutability results are unsuccessful. Based on the calculation results of groutability, the grouting mode in the sand layer of Beer-miao running tunnel is judged to be the fracture–compaction mode.

Literature	Calculation Results of Groutability	Judgment of Groutability	
Burwell (1958)	N=3.50<11,M=1.14<5	unsuccessful	
Mitchell (2006)	N = 3.50 < 11, M = 1.14 < 6	unsuccessful	
Akbulut and Saglamer (2002)	$N = 13.06 {\sim} 14.57 < 28$	unsuccessful	
Zhang (2017)	$N = 11.21 \sim 13.25 < 25$	unsuccessful	

Table 5. Calculation results of groutability.

4.4. Selection of Calculation Method of Grouting Diffusion Process and Grouting Reinforcement Effect

For the fracture–compaction mode, the coupling effect of grout flow and sand layer deformation should be considered in the calculation of the grouting process. Therefore, the calculation method for the fracture–compaction grouting diffusion process described in Section 2.4 was used to determine the grouting radius and shape of the grouted body under different grouting parameters and rheological properties of grout. The calculation method for fracture–compaction grouting reinforcement effect described in Section 2.5 was used to determine the macroscopic performance of the grouted body under different grouting parameters and different grouts.

4.5. Preliminary Grouting Design

4.5.1. Selection of Grout Type

Cement grout and cement–sodium silicate grout (C–S grout) are the two most widely used grouts in grouting projects. The advantage of cement grout is high later strength and good durability. C–S grout is typical quick-setting grout, consisting of two components. There is relatively high resistance in the grouting process for C–S grout.

In this paper, the two grouts are compared in aspects of diffusion and reinforcement characteristics to select the most suitable grout in this project. A typical calculation condition is selected based on the in situ situation of the project. The water–cement ratio of the cement grout is set to W/C = 0.8. C–S grout has a water–cement ratio of W/C = 0.8 and the volume ratio of two components of C:S = 1:1. The initial setting time of this C–S grout is 40~50 s. Comparison of the two grouts in aspects of basic performance, obtained by laboratory tests, is shown in Table 6.

	Rheological	Performance of Calculus						
	Parameters	Age	8h	1d	3d	7d	14d	28d
Cement $\tau_0 = 5.321 \text{ Pa}$ grout $\mu = 22.9 \text{ m Pa} \cdot \text{s}$		E_s/GPa	0.00	0.00	0.49	0.68	1.12	1.98
	$\tau_0 = 5.321 \text{ Pa}$	c/MPa	0.00	0.00	1.02	1.39	2.21	4.37
	$\mu = 22.9 \text{ m Pa} \cdot \text{s}$	φ	/	/	38.1	38.7	35.7	36.1
		$k/\mathrm{cm}\cdot\mathrm{s}^{-1}$	/	/		<1 × 10 ⁻⁸		
C-S grout τ_0 =		E_s/GPa	0.14	0.21	0.51	0.92	1.07	1.15
	$\tau_0 = 53.21 \text{ Pa}$	c/MPa	0.30	0.42	1.08	1.90	2.35	2.57
	$\mu = 0.229 \text{ Pa} \cdot \text{s}$	φ	38	39	37.5	38.1	35.2	34.3
		$k/\mathrm{cm}\cdot\mathrm{s}^{-1}$			$<1 \times 10^{-1}$	-8		

Table 6. Basic performances of cement grout and C-S grout.

Limited by the testing technique of rheological parameters of quick-setting grout, the relationship between viscosity, yield stress and reaction time of quick-setting grout cannot be measured efficiently for the moment. In this paper, with the aim to reflect the significant rheological difference between cement grout and C–S grout, viscosity and yield stress of C–S grout is set to ten times that of cement grout. Due to the approximate treatment of rheological parameters of C–S grout, it becomes possible to compare the two grouts in aspects of diffusion characteristics.

Grouting parameters are chosen based on similar grouting projects. For convenience of expression, grouting parameters of ultimate grouting design in this project are used. Injection rate is set to q = 83.4 L/min, and the influence range of grouting is set to D = 20 cm. The permeability coefficients of calculus of cement grout and C–S grout are all measured smaller than 1×10^{-8} cm·s⁻¹. In the calculation of grouting diffusion and effect, permeability coefficients of calculus of cement grout are set to 5×10^{-9} cm·s⁻¹.

In aspect of diffusion characteristics, the grouting process including the evolution of grouting pressure and the relationship between the fracture width and distance from the injection hole was calculated by the method presented in Section 2.4. Calculation results of the grouting process of two grouts are shown in Figure 14. As shown in Figure 14a,b, the fracture width of C-S grout is approximate twice that of cement grout with the same injection rate. Therefore, there is a thicker grout vein of C–S grout than cement grout. As shown in Figure 14c, due to low resistance in the grouting process of cement grout, the grouting radius of cement grout is bigger than that of C–S grout. The grouting radius of cement grout can exceed 15 m after about 30 min of grouting time. However, with the width of the excavation section of the tunnel being merely 7.0 m, the grouting radius of cement grout is much larger than the dimension of the target region of grouting. There is little effect of having a big grouting radius on improvement of the retained ratio of grout take in the target region of grouting. Inversely, grout would be wasted if using cement grout. As shown in Figure 14d, the grouting pressure of C–S grout is 0.3–0.5 MPa higher than that of cement grout. High grouting pressure is beneficial to improve compaction degree of sand layer, and better performance of sand layer would be acquired. C-S grout is better than cement grout in aspect of diffusion characteristics, owing to that thicker grout veins and higher compaction degree would be acquired if using C-S grout.



Figure 14. Comparison on diffusion characteristics of two grouts. (a) Fracture width (t = 15 min). (b) Fracture width (t = 60 min). (c) Grouting radius. (d) Grouting pressure.

Reinforcement effects of the two grouts are calculated under typical conditions that injection time is t = 60 min and interval of injection holes is L = 20 cm. Calculation results at the position of the injection hole are shown in Figure 15. As shown in Figure 15, the macroscopic performance of the grouted body is obviously anisotropic. In terms of the modulus of compression, cohesion and permeability coefficient, the performance of the grout body in a direction parallel to the grout veins is different from that in a direction perpendicular to the grout veins. The modulus of compression and permeability in directions perpendicular to the grout veins are smaller than that in directions parallel to grout veins, expressed as $E_{sv} < E_{sh}$ and $k_v < k_h$. Cohesion in directions perpendicular to grout veins is bigger than that in another direction, expressed as $c_v > c_h$. There is little difference between the internal friction angles in two directions, due to the internal friction angle of the grouted body increasing, the modulus of compression in a direction parallel to grout veins and cohesion in a direction perpendicular to grout veins and being approximately equal to that of the grout calculus. With the age of the grouted body increasing, the modulus of compression in a direction parallel to grout veins and cohesion in a direction perpendicular to grout veins and cohesion in a direction perpendicular to grout veins and cohesion in a direction perpendicular to grout veins increase markedly and other performance parameters do not obviously change.



Figure 15. Comparison on reinforcement effects of two grouts. (**a**) Modulus of compression. (**b**) Cohesion. (**c**) Internal friction angle. (**d**) Permeability coefficient.

As shown in Figure 15, in ages of 8 h~28 d, the modulus of compression in two directions of C–S grout is higher than that of cement grout with a gap of 7~30 MPa. Cohesion in two directions of C–S grout are higher than that of cement grout, with gap of 12~30 kPa in perpendicular direction and 1.5 kPa in parallel direction. The permeability coefficient in two directions of the C–S grout is 28~37% lower than that of cement grout. Although the long-term performance of cement grout calculus is better than C–S grout, C–S grout is better than cement grout in terms of reinforcement effects during early and

late ages of the grouted body, due to the thicker grout vein and higher compaction degree when using C–S grout.

Based on the analysis above, C–S grout is better than cement grout in aspects of diffusion characteristics and reinforcement effects. Therefore C–S grout is used in this project.

4.5.2. Determination of Grouting Parameters

Firstly, grouting parameters (injection rate, injection pressure and grout take), grouting reinforcement scope and arrangement of the injection holes are determined preliminarily. Secondly, the grouting diffusion process and reinforcement effect of the grouted body are calculated to acquire grouting radius, shape of the grouted body and performance parameters of the grouted body. Thirdly, deformation and stability of the surrounding rock are analyzed by numerical simulation software. Grouting parameters are continuously adjusted to meet the grouting design objective. The final determined grouting parameters in this project are as follows:

(1) Injection rate and stop criterion for grouting

Injection rate is determined at 83.4 L/min. Grout take of a single injection hole is 2 m^3 . Grout take is used as the single stop criterion to decide whether to end the grouting process. When grout take of a single injection hole reaches a value of 2 m^3 , grouting should be ended.

(2) Grouting reinforcement scope and arrangement of injection holes

As shown in Figure 16, grouting reinforcement scope is the area that extends 3 m outside the boundary of tunnel excavation. The lower boundary of grouting reinforcement scope is 1m below the roof arch of the tunnel. In every cycle for grouting reinforcement and excavation, reinforced length is 12 m and excavation length is 9 m. The leftover soil mass with a length of 3 m is used as sealing mass for grouting in the next cycle. There are three series of injection holes in grouting reinforcement scope, with eight, seven and eight injection holes for first, second and third series respectively. Longitudinal reinforcement ranges of injection holes are 0~5 m, 0~8.5 m and 0~12 m, respectively. In the second and third series of injection holes, segmental grouting technology is adopted and the segmental interval is set to 4 m. There are two and three grouting segments for the second and third series injection holes, respectively. The principle of arrangement of injection holes is to guarantee that the overall sand layer in the grouting reinforcement scope is compacted. So, the interval between the ends of the injection holes should be set at an appropriate value. The interval between the ends of the injection holes is determined at 2~2.5 m, with the corresponding average interval of all injection holes being L = 17.4 cm.



Figure 16. Design of injection holes (unit: mm). (a) Cross section. (b) Longitudinal section.

Calculation results of macroscopic performance parameters of the sand layer before and after the grouting operation are shown in Table 7. Compared to performance before the grouting operation, there is obvious improvement of performance parameters after the grouting operation, with the modulus of compression increasing by 194%, cohesion increasing by 219% and the permeability coefficient decreasing by 76%.

	Modulus of Compression	Cohesion	Internal Friction Angle	Permeability Coefficient
Before grouting operation	14.09 MPa	14.71 kPa	32.29°	$4.87 imes10^{-3}~{ m cm/s}$
After grouting operation	$E_{sv} = 28.8 \text{ MPa}$ $E_{sh} = 53.93 \text{ MPa}$	$c_v = 76.27 \text{ kPa}$ $c_h = 17.66 \text{ kPa}$	$arphi_v = 33.62^\circ$ $arphi_h = 33.4^\circ$	$k_v = 9.06 \times 10^{-8} \text{ cm/s}$ $k_h = 2.3 \times 10^{-3} \text{ cm/s}$
Average performance after grouting operation	41.37 MPa	46.97 kPa	33.51°	$1.15\times10^{-3}~{\rm cm/s}$

Table 7. Calculation results of macroscopic performance parameters of sand layer.

COMSOL Multiphysics, Stockholm Sweden, an FEM software, is used to analyze the tunnel stability after the grouting operation. A two-dimensional model is adopted in the numerical simulation. Boundary conditions and mesh generation of the calculation model are shown in Figure 17. Parameters of the overlying soil and base rock from the document of the engineering geological investigation are used in calculation. Parameters of the sand layer and reinforcement scope from Table 6 are used in calculation. Due to the fact that the anisotropic performance of the grouted body cannot be analyzed in COMSOL Multiphysics, the average performance parameters after the grouting operation in Table 6 are used as parameters of reinforcement scope. The Mohr–Coulomb criterion was used in numerical modeling.



Figure 17. Calculation model.

Calculation results of tunnel stability are shown in Figure 18. Vertical settlement of the tunnel after excavation is concentrated in the region of the roof arch. The maximum value of vertical settlement is 10.7 cm, which can be considered as a safe value. The plastic zone is represented by the red zone in Figure 18b, which is mainly distributed in the region near two spandrels of the tunnel. Most of the plastic zone is located within the scope of grouting reinforcement, and there is no extension of the plastic zone in the undisturbed sand layer. Therefore, the surrounding rock of the tunnel can be considered stable.



Figure 18. Calculation results of tunnel stability. (**a**) Settlement distribution (unit: m). (**b**) Distribution of plastic zone.

4.6. Inspection of Grouting Effectiveness

4.6.1. Dynamic Monitoring Data Analysis (p-q-t)

The injection rate was controlled at 83.4 L/min during grout injection. Three typical injection holes were selected to analyze variation of the grouting pressure. Comparisons on grouting pressure of the calculation results and monitored data are shown in Figure 19.



Figure 19. Comparison on grouting pressure (calculation results and monitored data).

As shown in Figure 19, grouting pressure is increasing during the injection. The increasing trend of grouting pressure of the calculation results is basically consistent with that of monitored data, with a relative error smaller than 40%. Grouting pressure of monitored data at injection hole 1–2 is basically equal to that of calculation results. However, grouting pressure of the monitored data at other two injection holes is higher than that of the calculation results. After many grouting operations, the sand layer has been compacted and reinforced effectively. Consequently, the anti-destruction performance of the injected medium becomes better. There is stronger resistance in the grouting process for subsequent grouting operation. As a result, grouting pressure becomes higher with increasing grouting frequency. Injection holes 1–2 are operated in early stage, and its grouting pressure of monitored data is basically equal to that of the calculation results. Injection holes 2–3 and 3–5 are operated later than injection hole 1–2, resulting in their pressure being higher than that of the calculation results.

4.6.2. Checkhole Investigation

Stratum integrity and permeability after the grouting operation can be acquired from checkholes. After the grouting operation, nine checkholes were drilled in grouting reinforcement scope 24 h later. There is no or little water flow in the checkholes after the grouting operation, with permeability coefficient smaller than 1×10^{-4} cm/s. Performance of impermeability of sand layer meets the requirement of water outflow in tunnel. Borehole televiewer was used before and after the grouting operation to intuitively inspect stratum integrity. Results of the borehole televiewer are shown in Figure 20. Before the grouting operation, stratum integrity is poor, with much crushed stone in the checkhole. After the grouting operation, stratum integrity is clearly improved with no crushed stone in the checkholes. The grouted body has acquired self-stability. The reinforcement effect of grouting on the sand layer is significant.



Figure 20. Results of borehole televiewer. (a) Before grouting operation. (b) After grouting operation.

4.6.3. Observation of Grouted Body Exposed in Tunnel Face

After excavation, the grouted body is exposed in the tunnel face as shown in Figure 21. The grouted body exists in the form of grout veins, proving that the judgment of grouting mode in this project is right. There are a large number of grout veins and no seepage of water in the sand layer. The sand layer is stable enough to guarantee excavation safety during the period from the blasting operation to completing primary support.



Figure 21. Grout veins exposed in tunnel face.

As shown in Figure 21, grout veins are approximately parallel with each other, consistent with the basic assumption of calculation model for fracture–compaction grouting.

4.6.4. Monitoring of Tunnel Deformation

As soon as the primary support is completed, monitoring points are installed in the tunnel. Monitored data of vertical settlement and horizontal convergence in three cross sections of the tunnel are shown in Figure 22. Vertical settlement and horizontal convergence increase with the continuing excavation. When the distance between the tunnel face and monitoring points exceeds 20 m, the monitored data tend to be stable gradually.

The final vertical settlement of tunnel is in the range of 5~8 mm. The final horizontal convergence of tunnel is in the range of 2~4 mm. Excavation stability of tunneling improves significantly after the grouting operation with deformation of tunnel in the safe range.



Figure 22. Monitored tunnel deformation. (a) Vertical settlement. (b) horizontal convergence.

Based on several inspection results, including analysis of dynamic monitoring data, analysis of checkhole investigations, observation of grouted body exposed in the tunnel face and monitoring of tunnel deformation, the grouting operation in Beer-miao running tunnel has achieved the desired results. The objective of this project to guarantee that engineering safety of tunnel construction is reached. Hence, the effectiveness of the quantitative grouting design method is verified.

5. Conclusions and Discussion

In this paper, a systematic grouting design method for the sand layer is presented. This method has achieved quantitative design in aspects of the judgment of the grouting mode, calculation of grouting range and reinforcement effects. To calculate the grouting process for the fracture–compaction grouting mode in sand, a calculation method is developed which considers the coupling effect of grout flow and sand compaction deformation. In order to acquire macroscopic performance parameters of the grouted body for the fracture–compaction grouting mode in sand layers, a calculation model is established which can connect macroscopic performance and individual performance of the grout vein, compacted sand and undisturbed sand.

The quantitative grouting design method put forward in this paper has been used in the sand grouting project of Beer-miao running tunnel in Qingdao metro line 2. The selection of grout types, determination of grouting parameters, grouting reinforcement scope and arrangement of injection holes were all completed based on the design method. After the grouting operation, several inspection approaches were carried out to evaluate the effectiveness of the grouting design. The checkhole investigation shows that stratum integrity is clearly improved with no crushed stone in the checkholes. There is a large number of grout veins and no seepage of water in the tunnel face. The final vertical settlement and horizontal convergence of t the unnel are both smaller than 10 mm, where the effectiveness of the quantitative grouting design is verified.

However, the grouting process in sand layers is complicated and perfect description of the grouted body in engineering conditions is yet to be fully realized. Many simplifications are made, such as constant rheological parameters of grout, evenly distributed injection holes and so on. In addition, the precise grouting device is able to provide the constant grouting take during the grouting practice, and how to evaluate the role of these grouting devices in grouting engineering is still an important issue. For further research, the above factors will be studied to further improve the quantitative grouting design method. **Author Contributions:** Conceptualization, L.Z.; methodology, L.Z.; software, Z.L. and D.S.; validation, Q.Z., D.W. and L.W.; formal analysis, Z.L.; investigation, Z.L. and L.Z.; resources, L.Z.; data curation, Z.L.; writing—original draft preparation, Z.L. and D.W.; writing—review and editing, Z.L. and L.Z.; visualization, Z.L.; supervision, Q.Z.; project administration, L.Z.; funding acquisition, Z.L. and L.Z. All authors have read and agreed to the published version of the manuscript.

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