Criterion of Grouting Pressure in Regional Advance Grouting Treatment to Prevent Water Disaster from Karst Aquifers in Coal Seam Floors

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ABSTRACT: The deep mining of coal mines in North China faces the serious threat of water inrush from karst aquifers in the coal seam floors, and regional advance grouting technology (RAGT) is an effective means to prevent and control such disasters. However, it is difficult to choose the grouting pressure during the implementation of RAGT, and excessive grouting pressure will lead to the splitting of karst fracture and reduce the grouting effect. In this study, based on the Bernoulli equation, the relationship between the ground grouting pressure and critical grouting pressure during grouting is established. Based on the Hoek–Brown (H-B) strength criterion and a fracture mechanics analysis of hydraulic fracturing, a theoretical equation of the critical grouting pressure for fracture splitting during grouting is obtained. The determination methods of the main parameters, such as the length of the fracture, internal friction angle, and H-B constant of the intact rock and geological strength index, and their effects on the critical grouting pressure, are discussed. The results show that the joint influence of the H-B constant and geological strength index of the intact rock is the key factor influencing the critical grouting pressure. The theoretical research results are applied to the Xujiazhuang limestone grouting reinforcement project of the floor of coal seam 11 in the Zhaoguan coal mine. The critical grouting pressure of the aquifer is determined to be 14.54 MPa, which guides the smooth implementation of the project.

1. INTRODUCTION

With the increase in the mining depth of coal resources, the threat of deep karst aquifers to coal mine production is becoming increasingly significant, and water inrush accidents in coal mine floors are common. Grouting reinforcement is an effective means to control the water inrush disaster of coal mine floors. In China, the grouting methods for water disaster prevention and control of coal measures in karst aquifers are mainly divided into three categories, namely, curtain grouting, underground grouting reinforcement, and regional advance grouting technology. Regional advance grouting technology is a grouting reinforcement technology. After comprehensively considering factors such as the mine working face and rock stratum distribution, structural characteristics, and drilling construction efficiency, a well pad is arranged at a suitable position on the ground, and vertical wells are first constructed. Then, these wells are further drilled so that they enter the target aquifer at a certain angle and become nearly horizontal at a certain depth. Using the branching characteristics of horizontal drilling, each borehole is drilled along the target rock stratum, with bedding-directional branch boreholes constructed at the appropriate positions; each borehole track is constructed so that it crosscuts the water storage spaces and water diversion channels of the aquifer as much as possible. Then, the fractures are blocked by continuous high-pressure grouting to achieve the purpose of effectively transforming the aquifer. Curtain grouting and underground grouting reinforcement have the defects of a small treatment range and low filling efficiency. In the past 10 years, the concept of regional advance grouting technology by means of nearly horizontal drilling technology has been proven to be an effective way to prevent the water disaster of coal seam floors, making regional advance grouting technology gradually become the first choice for the...
prevention and control of top and bottom floor limestone water hazards.

Since 2008, the Jizhong energy group has carried out an attempt to advance the treatment of the water damage area of a karst aquifer in a coal seam floor and has used the ground-based multibranch directional drilling technology to treat an Ordovician limestone aquifer or a thin limestone in a coal measure basement. Since then, regional grouting treatment projects have been carried out in the Fengteng mining area, Huaing mining area, Huaihe mining area, and other mining areas. Dong et al. and Zhao et al. generalized the grouting transformation mode and selection criteria of floor aquifers suitable for the geological and hydrogeological conditions of North China-type coalfields, which provides theoretical support for the development of regional advance grouting technology in China. Zhang et al., Wu et al., and Zheng et al. discussed the key technologies and research directions to guide regional governance of mine water disasters, put forward the problems to be solved in drilling and grouting technology, improved its technical system, optimized the grouting transformation process, and expanded the application scope of this technology. In the revision of the detailed rules for water prevention and control in coal mines performed in 2018, recently proposed technology was popularized and suggested for water disaster control and has since developed rapidly. At present, scholars and engineers at home and abroad have mainly studied the grouting technology and drilling equipment of regional advance grouting technology under specific conditions; however, in the advance treatment engineering and research of the limestone floor area of coal measure strata, there is less information on the relevant factors affecting the effect of limestone coal seam floor grouting engineering, in which the grouting pressure control is an important index to ensure the effect of limestone floor grouting reinforcement engineering for coal seams. As an important parameter of grouting engineering design, grouting pressure plays an obvious role in slurry diffusion, fracture filling, and the grouting effect. If the grouting pressure is too low, the filling and cementation of the rock fractures will be incomplete; if the grouting pressure is too high, the slurry diffused to the fracture boundary will make the fracture hydraulically split, causing the fracture to expand at the tip. Therefore, it is necessary to determine an appropriate grouting pressure in the early stage of grouting engineering design.

Taking the Zhaoguan coal mine as an example, based on an engineering geological survey and the relationship between ground grouting pressure and critical grouting pressure, this paper analyzes a theoretical equation for determining the critical grouting pressure in karst fracture by using the theory of fracture mechanics, determines the different parameter values of the standard equation for grouting completion, and guides the grouting project of the Zhaoguan coal mine. The results have certain guiding significance for the practice of similar grouting projects.

2. METHODOLOGY

In this paper, the conceptual model of fracture grouting using directional drilling from the surface is established. Based on the Bernoulli energy equation, the relationship between the ground grouting pressure and critical grouting pressure is obtained after comprehensively considering the influence of factors such as the pressure loss caused by the movement of the cement grout in the cement grout transportation pipeline and the grouting hole and cement grout self-weight pressure. Through the fracture mechanics analysis of the Hoek–Brown (H-B) strength criterion and hydraulic fracturing, a theoretical calculation equation of the critical grouting pressure of type II fracture grouting based on the H-B strength criterion is obtained.

2.1. Conceptual Model of Directional Drilling Fracture Grouting. For the grouting of a fractured rock mass with this technique, the borehole is constructed from the ground. When the borehole intersects a water-conducting fracture, the cement grout moves and diffuses under the drive of grouting pressure. For each branch hole constructed for directional drilling grouting, the final borehole shape is an “L”, and the fractures exposed in the nearly horizontal section of the branch hole have a different orientation. For an area where any grouting project is located, the development of structural fractures in the rock stratum is directional, along one or several dominant directions. A conceptual model of fracture grouting by directional drilling from the surface is established in this paper (Figure 1).

![Figure 1. Schematic diagram of ground directional grouting.](https://doi.org/10.1021/acsomega.2c03450)

In Figure 1, \( p_{\text{hol}} \) is the ground grouting pressure, MPa; \( p_{\text{c}} \) is the critical grouting pressure, that is, the grouting pressure at the intersection of the borehole and fracture, MPa; \( p_{\text{w}} \) is the groundwater pressure in the fracture, MPa; \( p_{\text{f}} \) is the pressure of the cement grout diffusion front, MPa; \( H \) is the burial depth of the directional drilling grouting section, m; and \( r \) is the cement grout diffusion distance in the fracture, m.

2.2. Analysis of Grouting Pressure Based on the Bernoulli Equation. Assuming that the flow of cement grout in the grouting hole is approximately a continuous and uniform constant flow, the flow of cement grout is an incompressible flow. In the process of grouting, the viscosity effect and bleed effect of cement are ignored, and the mixing effect of cement slurry and crevice water is not considered. In Figure 1, sections A and B are taken to represent the cement grout state at the ground and the bottom of the grouting hole, respectively. Based on the Bernoulli energy equation, the following equation is established:

\[
p_A + \frac{u_A^2}{2g} + h = p_B + \frac{u_B^2}{2g} + p_w + h_{\text{los}}
\]

where \( p_A \) and \( p_B \) are the ground grouting pressure and critical grouting pressure, respectively; \( u_A \) and \( u_B \) are the cement grout flow velocities of the two sections, m/s; \( h_{\text{los}} \) is the head loss of the cement grout in the grouting hole, m; \( p_A \) and \( p_B \) are
the densities of the cement grout and groundwater, kg/m³; and $g$ is the gravitational acceleration, m/s².

The flow of cement grout in the borehole is continuous, so $u_A = u_p$. After calculation, the head pressure loss during the movement of the cement grout in the borehole is negligible. Based on the above analysis, eq 1 can be rewritten as

$$P_A = P_B + P_{\text{gro}} - \rho g H$$  

(2)

Obviously, in eq 2, $P_A$ is the ground grouting pressure $p_{\text{hol}}$ and $P_B + P_{\text{gro}}$ is the critical grouting pressure $p_{\text{gro}}$. Therefore, in the process of directional hole grouting, the relationship between the ground grouting pressure and critical grouting pressure can be approximately expressed by the following equation:

$$p_{\text{gro}} = p_{\text{hol}} + \rho g H$$  

(3)

2.3. Determination Theory of Grouting Pressure Based on Fracture Mechanics. 2.3.1. Hoek–Brown Strength Criterion. In this paper, it is assumed that the fractures of the surrounding rock are relatively closed, their cross sections are regular ellipses, and the grouting borehole passes through the center of the fracture. The grouting process in the fracture is the process of cement grout displacing groundwater. When the cement grout diffuses to the fracture boundary, all the groundwater in the fracture passes through the center of the fracture. The grouting process in the fracture is the process of cement grout displacing groundwater. When the cement grout diffuses to the fracture boundary, all the groundwater in the fracture passes through the center of the fracture. The grouting process in the fracture is the process of cement grout displacing groundwater. When the cement grout diffuses to the fracture boundary, all the groundwater in the fracture passes through the center of the fracture.

In the process of grouting, with an increasing critical grouting pressure, the cement grout that diffuses to the fracture boundary will make the fracture hydraulically split, resulting in the expansion of the fracture at the tip. Therefore, based on the theory of fracture mechanics, the mechanism of hydraulic fracturing of fracture under the grouting pressure is analyzed, and the upper limit of the grouting pressure is determined to guide the determination of the critical grouting pressure.

E. Hoek and E. T. Brown proposed a strength criterion for rock samples, and the yield condition is

$$\sigma_1 = \sigma_3 + \sigma_4 \left( m_b \sigma_i/a_i + s \right)$$  

(4)

where $m_b$, $s$, and $a$ are used to describe the basic characteristics of the rock materials and are semi-empirical parameters, $\sigma_1$ is the maximum principal stress, $\sigma_3$ is the minimum principal stress, and $\sigma_i$ is the uniaxial compressive strength of the rock.

In practice, these parameters are obtained according to the geological strength index (GSI) and used to calculate eq 4. In the H-B strength criterion, the relationships between coefficients $m_b$, $s$, and $a$ in eq 4 and GSI are as follows

$$m_b = m \exp \left( \frac{GSI - 100}{28 - 14D} \right)$$  

(5)

$$s = \exp \left( \frac{GSI - 100}{9 - 3D} \right)$$  

(6)

$$a = \frac{1}{2} + \frac{1}{6} \left[ \exp \left( \frac{-GSI}{15} \right) - \exp \left( \frac{-20}{3} \right) \right]$$  

(7)

where $D$ is the disturbance coefficient of the rock mass excavation, $m$ is the H-B constant of the intact rock block, $s$ is the fragmentation degree of the rock mass, and $a$ is the parameter related to the integrity of the rock mass.

For the rock mass, eqs 5–7 mainly describe the geological conditions of the site through the GSI, and $s$ and $a$ are calculated from the GSI and $D$ values, respectively. Then, $m_b$ and $m_i$ are determined by the indoor triaxial testing on rock samples, so that the rock mass strength finally can be determined according to eq 4 with the support of crustal stress test data.

2.3.2. Fracture Mechanics Analysis of Hydraulic Fracturing. In the process of grouting in a fractured rock mass, when the grouting hole intersects a fracture, the cement grout gradually fills the rock fracture. With the continuous increase in grouting pressure, the splitting phenomenon at the fracture tip will be induced, and the cement grout flow will increase because of the continuous expansion of the fracture. The fracture action at the fracture tip leads to two results: (1) When the expansion range of the fracture grouted with cement grout is limited, it can communicate with the unexposed fractures in the borehole and improve the grouting reinforcement effect of the rock mass fracture network around the grouting hole. (2) If the grouting pressure rises continuously and the expansion of the grouted fracture is uncontrollable, the fracture will expand excessively, which causes a large amount of cement grout loss. It can be seen that the selection of the critical grouting pressure is very important. In this paper, the theoretical value of the critical grouting pressure is formulated based on the criterion of whether it can cause a fracture to expand excessively.

Hydraulic fracturing is a process in which rock fractures expand intermittently under the action of high head pressure and then further open after they are connected with each other. Therefore, Zou et al. established the fracture mechanics model shown in Figure 2 to analyze the critical head pressure of hydraulic fracturing. On this basis, the mechanical influence of fracture grouting pressure in a fractured rock mass is studied, and the critical fracture pressure is determined.

![Figure 2. Fracture mechanics analysis model of a fractured rock mass.](https://doi.org/10.1021/acsomega.2c03450)
\[
\begin{align*}
\sigma_c &= \frac{\sigma_1 + \sigma_3}{2} - \frac{\sigma_1 - \sigma_3}{2} \left( \frac{\partial\sigma_1}{\partial s} - 1 \right) \\
\tau &= (\sigma_1 - \sigma_3) \sqrt{\frac{\partial\sigma_1}{\partial s}} + 1 \\
\frac{d\sigma_1}{d\sigma_3} &= 1 + am_b \left( m \frac{\sigma_1}{\sigma_3} + s \right)^{a-1}
\end{align*}
\]

(8)

where \(\sigma_1\) and \(\sigma_3\) are the maximum and minimum principal stress of the rock mass at its yield point, respectively; \(s_0\) is the normal stress on the fracture surface; and \(\tau\) is the shear stress on the fracture. Equation 8 shows the normal stress and shear stress for the fracture without the action of fluid pressure inside the fracture.

In this paper, it is assumed that the grouted fracture is relatively closed and the grout completely occupies the fracture space after grouting. At this time, the fracture simultaneously bears the effects of the internal grouting pressure and external surrounding rock water pressure, so the fluid pressure inside the fracture is the difference between the two. When the grouting pressure in the grouted fracture is equal everywhere, the grouting pressure is the critical grouting pressure. At this time, the fluid pressure inside the fracture is \(p_f = p_{gr} - p_w\) and the normal stress and shear stress on the fracture can be expressed as

\[
\begin{align*}
\sigma_n &= \frac{\sigma_1 + \sigma_3}{2} - \frac{\sigma_1 - \sigma_3}{2} \left( \frac{\partial\sigma_1}{\partial s} + m \frac{\sigma_1}{\sigma_3} + s \right)^{a-1} \\
\tau &= -\left( \sigma_1 - \sigma_3 \right) \frac{1 + m \frac{\sigma_1}{\sigma_3} + s}{2 + am_b \left( m \frac{\sigma_1}{\sigma_3} + s \right)^{a-1}} \\
&\quad \left( \sigma_1 - \sigma_3 \right) \frac{1 + am_b \left( m \frac{\sigma_1}{\sigma_3} + s \right)^{a-1}}{2 + am_b \left( m \frac{\sigma_1}{\sigma_3} + s \right)^{a-1}}
\end{align*}
\]

(9)

The most important concern in engineering is the fracture propagation condition. In this paper, the approximate criterion is selected

\[
\begin{align*}
K_1 + K_{II} &= K_{IC} \\
K_1 &= -\sigma_0 \sqrt{\pi l} \\
K_{II} &= K_{IC} = \tau \sqrt{\pi l}
\end{align*}
\]

(10)

where \(K_1\) is the propagation criterion of a mode I fracture, \(K_{II}\) is the propagation criterion of a mode II fracture, \(K_{IC}\) is the type I fracture toughness, and \(K_{ICB}\) is the type II fracture toughness.

According to the analysis of geological conditions, the fractures in a karst aquifer are affected by regional crustal stress and undergo large normal compressive stress. Therefore, the fracture grouting problem is a type II fracture problem. A fracture will be closed under the action of compressive stress. A closed fracture can transfer normal stress and shear stress. At this time, the effective shear stress on the fracture is

\[
\tau' = \tau - \sigma_0 \tan \varphi
\]

(11)

where \(\varphi\) is the internal friction angle of the rock at the fracture surface.

Assuming that the closing force of the fracture is 0, substituting the effective shear stress \(\tau'\) on the fracture into the \(K_{II}\) expression gives

\[
p_{IC} = \frac{\sigma_1 + \sigma_3}{2} - \frac{\sigma_1 - \sigma_3}{2} \left( \frac{1 + m \frac{\sigma_1}{\sigma_3} + s}{2 + am_b \left( m \frac{\sigma_1}{\sigma_3} + s \right)^{a-1}} \right)
\]

\[
- \left( \frac{K_{IC} \sqrt{\pi l}}{\sigma_1 - \sigma_3} + \frac{1}{\tan \varphi} \right) + p_w
\]

Thus far, the \(p_{gr \cdot c}\), theoretical calculation equation of critical grouting pressure for type II fracture grouting based on the H-B strength criterion is obtained.

\[
p_{gr \cdot c} = \frac{\sigma_1 + \sigma_3}{2} - \frac{\sigma_1 - \sigma_3}{2} \left( \frac{1 + m \frac{\sigma_1}{\sigma_3} + s}{2 + am_b \left( m \frac{\sigma_1}{\sigma_3} + s \right)^{a-1}} \right)
\]

\[
- \left( \frac{K_{IC} \sqrt{\pi l}}{\sigma_1 - \sigma_3} + \frac{1}{\tan \varphi} \right) + p_w
\]

(13)

3. EXPERIMENTAL SECTION

3.1. Study Area. The study area is located in Shandong Province, China. The site occurs between 36°28′00″N–36°33′30″N latitude, and 116°20′00″E–116°36′30″E longitude and covers an area of approximately 65 km², which is spread over 9 km from north to south and along a distance of 7.2 km from east to west.

3.2. Geological Conditions. The study area is located in the northern coalfield along the Yellow River, which is a Carboniferous Permian coalfield of the North China type. The strata developed in the area from old to new include the middle Lower Ordovician system (O₂₁⁺), Benxi Formation of the middle Carboniferous System (C₂₃.), Taiyuan Formation of the upper Carboniferous System (C₂₄.), Shanxi Formation of the lower Permian (P₁.), Lower Shihezi Formation of lower Permian (P₂.), Upper Shihezi Formation of upper Permian (P₃.), Neogene system (N), and Quaternary system (Q). The coal-bearing strata in the mine field are the Taiyuan Formation and Shanxi Formation of the Carboniferous-Permian strata, and the main coal seams are coal seams 7, 11, and 13, as shown in Figure 3.

3.3. Hydrogeological Conditions. According to the analysis of the main water-filling factors in coal seam mining,
Karst aquifer water is the main water source of the Zhaoguan coal mine, and the Xujiazhuang limestone aquifer of the Benxi Formation and Ordovician limestone aquifer are developed in the floor of coal seams 11 and 13. The average thickness of the Xujiazhuang limestone of the Benxi Formation in the floor is 8.49 m, the highest water level elevation is +31.93 m, and the unit water inflow is 0.0085 L/s·m. It is a karst fracture-confined aquifer with a strong water yield, and the runoff recharge cycle conditions are good. The thickness of the Ordovician limestone can reach approximately 800 m, and the unit water inflow is 0.1517 L/s·m; it is a karst fracture-confined aquifer with a generally strong water yield. This Ordovician limestone has a close hydraulic connection with the Xujiazhuang limestone of the Benxi Formation and is also a potential threat to the mining of lower coal seams. Coal seams 11 and 13 are located 36 and 29 m from the Xujiazhuang limestone of the Benxi Formation, respectively. According to the detailed rules for water prevention and control in coal mines, the corresponding calculated water inrush coefficients are 0.12 and 0.15 MPa/m. In addition, according to the supplementary hydrogeological exploration results of the Zhaoguan coal mine, there is a vertical hydraulic connection between the Xujiazhuang limestone aquifer of the Benxi Formation and the Ordovician limestone aquifer. Therefore, coal seams 11 and 13 face a serious threat of water inrush during mining, and regional grouting treatment should be carried out before coal resource mining.

To solve the mining problem of the lower coal group, grouting treatment of the Xujiazhuang limestone aquifer of the Benxi Formation of the Zhaoguan coal mine is carried out. An area of 600 m × 600 m east of the mining area is selected as the experimental area. Four well pads are arranged at favorable positions on the ground of the treatment area, and a total of 48 boreholes are constructed to carry out grouting treatment of the Xujiazhuang limestone aquifer of the Benxi Formation. The drilling construction design is shown in Figure 4. The drilling is divided into three sections: straight hole section, directional deflecting hole section, and directional horizontal section. The diameter of the straight hole section is 311 mm, the diameter of the directional deflecting hole section is 216 mm, and the diameter of the directional horizontal hole section is 154.2 mm. The grouting material is a cement single-liquid slurry, with ordinary silicate PO32.5R cement with a density of 1.3 × 10^3 kg/m^3. The grouting pressure control is selected as the end standard of the project grouting. At a position of 50 m from the wellhead, the boreholes enter the Xujiazhuang limestone aquifer of the Benxi Formation, pass through the layers, and crosscut the fractures of the limestone. Then, grouting and plugging are carried out. There, the Xujiazhuang limestone aquifer of the Benxi Formation transforms into water-resisting layers that block the horizontal and vertical water channels.

### 3.4. Main Parameters and Their Determination Methods

According to the analysis of eq 13, the main factors affecting the hydraulic fracturing of rock fractures are the crustal stress, geological strength index (GSI), rock uniaxial compressive strength, fracture toughness, and rock internal friction angle.

#### 3.4.1. Uniaxial Compressive Strength and Internal Friction Angle

The uniaxial compressive strength and internal friction angle of rock can be obtained through indoor rock mechanics tests, which are carried out on the Zhaoguan coal mine rock. Based on the measured data from the Zhaoguan coal mine, it is determined that the saturated uniaxial compressive strength of the Xujiazhuang limestone aquifer of the Benxi Formation in the test area is 79 MPa and that the internal friction angle is 37°.

#### 3.4.2. Rock Mass Disturbance Parameter

The rock mass disturbance parameter (D) reflects the disturbance degree of blasting damage to a jointed rock mass. For an undisturbed rock mass, D = 0; for a highly disturbed rock mass, D = 1. The Xujiazhuang limestone aquifer of the Benxi Formation in the experimental area was not artificially disturbed before grouting, so the rock mass disturbance parameter is D = 0.

#### 3.4.3. Hoek–Brown Constant of an Intact Rock Block

Hoek proposed a method to obtain the H-B constant mi of a rock by rock uniaxial tests or triaxial tests in the laboratory. When there are few indoor test data, the mi value can be estimated according to the data types presented in Table 1.
Figure 4. Directional drilling construction schematic diagram.

Table 1. Determination of the $m_i$ Value

| rock type      | rock formation/structure | coarse grain       | medium grain        | fine grain           | very fine grain       |
|----------------|--------------------------|--------------------|---------------------|----------------------|-----------------------|
| sedimentary   |                           | conglomerate 21 ± 3 | conglomerate        | siltstone 7 ± 2      | clay rock 4 ± 2        |
| rock          |                          | breccia 19 ± 5     |                      | graywacke 18 ± 3     | shale 7 ± 2            |
|                |                          | coarse-grained limestone 12 ± 3 |                     | microcrystalline limestone 9 ± 2 | dolomite 9 ± 3         |
|                |                          |                     | bright limestone 10 ± 2 | plaster 8 ± 2        |                       |
|                |                          |                     |                     |                      |                       |
| metamorphic   |                          | marble 9 ± 3        | angular shale 19 ± 4 | quartze 20 ± 3       |                       |
| rock          |                          |                     | metamorphic sandstone 19 ± 3 |                    |                       |
|                |                          |                     | gneiss 28 ± 5        | amphibolite 26 ± 6   |                       |
| igneous       | plactic                  | granite 32 ± 3      | migmatite 29 ± 3     | schist 12 ± 3        | slate 7 ± 4            |
| rock          | rocks                    | granodiorite 29 ± 3 | nesoschist 28 ± 5    | phyllite 7 ± 3       |                       |
|                |                          | diorite 25 ± 5      | gneiss 28 ± 5        |                       |                       |
|                |                          | gabbro 27 ± 3       | granite 32 ± 3       |                       |                       |
|                |                          | coarse-grained basalt 16 ± 5 |                     |                       |                       |
|                |                          | long rock 20 ± 5     | gabbro 27 ± 3        |                       |                       |
|                |                          |                      | coarse-grained basalt 16 ± 5 |                     |                       |
|                |                          |                      |                     |                       |                       |
| hypabyssal    | plactic                  | porphyry 20 ± 5     | rhyolite 25 ± 3      | diabase 15 ± 5       | peridotite 25 ± 2      |
| rock          | rocks                    |                      | basalt 25 ± 5        |                       |                       |
|                |                          |                      | volcanic breccia 19 ± 5 |                       |                       |
|                |                          |                      |                     |                       |                       |
| extrusive     |                          |                      |                      |                       |                       |
| rock          | lava                     |                    |                      |                       |                       |
|                |                          |                      |                      |                       |                       |
| pyroclastic    |                          |                      |                      |                       |                       |
According to the existing test data of the H-B constant $m_i$ and Table 1, the Xujiazhuang limestone of the Benxi Formation in the test area has fully developed crystal cleavage, and the $m_i$ value is comprehensively determined to be 7.

### 3.4.4. Geological Strength Index

GSI is a parameter characterizing the structural characteristics of rock masses and structural surface characteristics (roughness, weathering degree, and fillings). Based on many rock mass engineering experiences, Hoek put forward the GSI recommendation.
table, as shown in Figure 5. In the GSI system, the structural characteristics of the rock mass are divided into six grades, and the structural plane conditions are divided into five grades.

According to the leakage of cement slurry in the horizontal section of a completed borehole near the study area, only a few structural planes, with a large spacing, are distributed in the study area. According to the geological data actually observed underground and the water inflow information exposed by the grouting hole data in the study area, the karst fractures of the Xujiazhuang limestone aquifer of the Benxi Formation in the study area are underdeveloped; therefore, the rock mass structure in the study area is relatively intact. Generally, after a long-term dissolution of groundwater, karst fracture surfaces in a limestone aquifer become smooth and weathered. Therefore, the characteristics of karst fracture surfaces are classified as general. According to Figure 5, the GSI of the karst aquifer in the studied mine GSI is equal to 55–80. Due to the weathering of limestone to a certain extent, the GSI value should be small, so it is taken as 60 in this paper. Thus, \( m_{0}, s, a, \) and other parameters are calculated with this information.

3.4.5. Crustal Stress. In general, to describe the crustal stress of a mine, it is necessary to carry out crustal stress tests. The main test methods are the stress relief method and hydraulic fracturing method. If there are no measured data in a study area, we can refer to the crustal stress test data of adjacent areas to understand the crustal stress distribution in the mine. A crustal stress test has not been carried out in either the Zhaoguan mine field or adjacent mines, and there is a lack of measured data in the study area. Referring to the existing research results, this paper analyzes the stress state of the Xujiazhuang limestone aquifer of the Benxi Formation in the study area. Li et al.\(^{46}\) collected 181 groups of in situ stress test data in Shandong Province (the distribution of in situ stress test points is shown in Figure 6). Figure 6 shows that the horizontal stress in Shandong is dominant within the measured depth range (the maximum depth is more than 1100 m), which belongs to a typical type of the tectonic stress field. The dominant direction of the maximum horizontal principal stress is thus NWW-SEE followed by NEE-SWW.

By fitting the variation relationships of the maximum horizontal principal stress, the minimum horizontal principal stress, and the vertical principal stress with depth in Shandong, the following relationships can be obtained

\[
\begin{align*}
\sigma_{h} &= 0.0242H + 9.4269 \quad (R^2 = 0.870) \\
\sigma_{v} &= 0.018H + 3.830 \quad (R^2 = 0.863) \\
\sigma_{i} &= 0.0258H + 0.563 \quad (R^2 = 0.889)
\end{align*}
\]

(14)

where \( \sigma_{h} \) is the maximum horizontal principal stress, MPa; \( \sigma_{v} \) is the minimum horizontal principal stress, MPa; \( \sigma_{i} \) is the vertical principal stress, MPa; \( H \) is the depth, m; \( m_{0} \) and \( R \) is the correlation coefficient.

According to eq 14 and the subsurface depth data of the grouting target layer, the crustal stress state of the Xujiazhuang limestone aquifer of the Benxi Formation is calculated. The average burial depth of the Xujiazhuang limestone aquifer is 480 m. The crustal stress component at the burial depth of this limestone is calculated according to eq 14, and the results are shown in Table 2. Table 2 shows that the maximum principal stress in the study area is the maximum horizontal principal stress, the minimum principal stress is the minimum horizontal principal stress, and the intermediate principal stress is the vertical principal stress.

3.4.6. Fracture Toughness. According to the analysis of geological conditions, the fractures in the karst aquifer in the study area are affected by regional crustal stress and thus produce a high normal compressive stress. Therefore, the fracture grouting problem is a mode II fracture problem. Fracture toughness is an important parameter in rock fracture mechanics. Fracture toughness represents the ability of rock materials to resist fracture propagation or the generation of new fracture surfaces and includes three basic modes: mode I, mode II, and mode III. Mode I fracture toughness (\( K_{Ic} \)) is required for the calculation of fracture grouting pressure. Mode I fracture toughness (\( K_{Ic} \)) reflects the ability of rock to resist tensile failure, and it is the most commonly used fracture toughness index.

Based on a large number of test data, Bao et al.\(^{11}\) deeply discussed the relationship between mode I fracture toughness (\( K_{Ic} \)) and rock uniaxial compressive strength, and this relationship can be fitted for different lithology conditions, as shown in Table 3.

| lithology | empirical equation | \( R^2 \) |
|-----------|--------------------|---------|
| sandstone | \( \sigma_{c} = 90.34 K_{Ic} \) | 0.85 |
| limestone | \( \sigma_{c} = 92.64 K_{Ic} \) | 0.85 |
| granite | \( \sigma_{c} = 95.26 K_{Ic} \) | 0.60 |

Therefore, based on the regression equation of limestone in Table 3 and the uniaxial compressive strength of Xujiazhuang limestone of the Benxi Formation in the study area, the mode I fracture toughness (\( K_{Ic} \)) of the Xujiazhuang limestone aquifer of the Benxi Formation is calculated to be 0.853 MPa·m\(^{1/2}\).

3.4.7. Hydrostatic Pressure of Aquifer. According to the supplementary hydrogeological survey and advance geophysical exploration data in the study area, it is comprehensively determined that the water pressure of the Xujiazhuang limestone aquifer of the Benxi Formation in the study area is 4.9 MPa.

3.5. Parameter Analysis. 3.5.1. Fracture Length. According to eq 13 and the determined relevant parameters, the relationship between fracture length (\( 2l \)) and critical splitting grouting pressure is shown in Figure 7. It can be seen from Figure 7 that there is a nonlinear relationship between the fracture length and the critical splitting grouting pressure. With the increase in the fracture length, the critical grouting pressure also gradually increases, and the growth rate gradually decreases. When the fracture length does not exceed 60 m, the growth rate of the critical grouting pressure decreases. In general, the longer the fracture is, the better the integrity of the rock mass. In contrast, the fracture length in a more broken rock mass is generally shorter. Therefore, with the increase in the fracture length, the integrity of the rock mass will improve,
the tensile stress and compressive shear stress to be overcome
by fracture expansion will continue to increase, and the
hydraulic splitting of the fracture is much more difficult,
resulting in the increase in critical grouting pressure.

3.5.2. Internal Friction Angle of Rock. According to eq 13
and the determined relevant parameters (fracture length is 60
m), the relationship between the internal friction angle and the
critical splitting grouting pressure of the rock is shown in
Figure 8. Figure 8 shows that with the increase in the internal
friction angle of the rock, the critical splitting grouting pressure
increases nonlinearly. The greater the friction angle of the
fracture, the greater the resistance needed to overcome the
fracture strength and make the slurry flow. Therefore, a greater
grouting pressure is needed.

3.5.3. Geological Strength Index. According to eq 13
and the determined relevant parameters (fracture length is 60 m),
the relationship between the GSI and the critical splitting
grouting pressure is shown in Figure 9. The GSI has a
nonlinear relationship with the critical splitting grouting
pressure. With an increase in GSI, the weathering degree of
the rock becomes weaker and the fillings in the fracture
decreases. The grouting pressure required for fracture splitting
decreases because the nonlinear strength is affected by the GSI.

3.5.4. Hoek–Brown Constant of Intact Rock Block.
According to eq 13 and the determined relevant parameters
(fracture length is 60 m), the relationship between the H-B
constant $m_i$ of an intact rock block and the critical splitting
grouting pressure is shown in Figure 10. Figure 10 shows that
with increasing $m_i$, the critical splitting grouting pressure first
decreases and then increases and reaches a minimum when $m_i$
is approximately 20. This trend arises because the nonlinear
strength parameters are affected by the parameter $m_i$. When
the $m_i$ is approximately 20, the influence of each parameter on
the splitting grouting pressure is the smallest. With the
continuous increase in $m_i$, the integrity of the rock improves,
and the critical splitting grouting pressure increases gradually.

For the same rock, to study the influence of the rock mass
structure on the grouting pressure required for fracture
splitting, under the joint action of parameters GSI and $m_i$,
the variation trend of the grouting pressure required for fracture splitting is shown in Figure 11. Figure 11 shows that for the fractured rock mass in the grouting project, the splitting grouting pressure decreases first and then increases with the simultaneous increase in parameters GSI and \( m_i \). However, the splitting grouting pressure of the fractured rock mass in the grouting project decreases significantly only when \( m_i \) is very small, and its change range is more significant when GSI is very large.

In summary, the main parameters required to calculate the critical splitting grouting pressure in the study area are determined, as shown in Table 4.

Table 4. Parameters Required for Calculating the Critical Splitting Grouting Pressure in Fractures of Aquifers

| parameters required for calculation | results for the limestone aquifer of the Xujiazhuang Formation |
|------------------------------------|---------------------------------------------------------------|
| \( \sigma_1 \)/MPa                  | 21.04                                                         |
| \( \sigma_2 \)/MPa                  | 12.47                                                         |
| \( \sigma_3 \)/MPa                  | 79                                                           |
| \( \phi \)/°                        | 37                                                           |
| GSI                                | 60                                                           |
| \( m_i \)                          | 7                                                            |
| \( K_i \)/MPa·m\(^{1/2}\)          | 0.853                                                        |
| \( m_s \)                          | 1.68                                                         |
| \( s \)                            | 0.01                                                         |
| \( a \)                            | 0.5                                                          |
| \( l/m \)                          | 30                                                           |

3.6. Grouting Completion Standard of the Ground Directional Grouting Reconstruction Project. The critical splitting grouting pressure of fracture tip expansion is calculated using eq 13 as the upper limit of the fracture grouting pressure. According to the relationship between the critical grouting pressure \( p_{gro} \) and the ground grouting pressure \( p_{hol} \), the theoretical value of the end pressure of grouting of the Xujiazhuang limestone aquifer directional grouting reconstruction project of the Benxi Formation is calculated.

Taking the calculation parameters in Table 4 into eq 13, the value of critical grouting pressure \( p_{gro\cdot c} \) is 14.54 MPa. Taking the calculated results into eq 3, the theoretical value of the end pressure of grouting \( p_{hol\cdot c} \) is 8.43 MPa, that is, during the grouting reinforcement project of the Xujiazhuang limestone aquifer in the Benxi Formation, the grouting pressure should be controlled at approximately 8.5 MPa.

4. RESULTS

The Zhaoguan coal mine adopts the above theoretical value of grouting reinforcement to strengthen the Xujiazhuang limestone aquifer of the Benxi Formation in the experimental area. Four drilling areas are designed for the grouting reinforcement project, with a total of 8 main holes and 40 branch holes, and the distance between branch holes is 60 m. These results, combined with the observations of the actual grouting project in the Zhaoguan coal mine, show that the change in the ground grouting pressure presents two forms. Taking the typical boreholes in the study area as an example, this paper analyzes these two pressure curves:

As shown in Figure 12, when the grouting project of the first hole has been implemented for 128 h, the end pressure of grouting is 8.5 MPa, and the cumulative amount of injected cement grout is 1193.00 m\(^3\). The grouting pressure of this hole rises slowly and reaches the end pressure of grouting. According to the curve analysis, the fracture range near the hole is small, and the connectivity with the surroundings is poor. While the grouting rate decreases, the grouting pressure maintains a slow upward trend, indicating that the fracture filling rate around the grouting point is high, the grouting operation is difficult, but the grouting effect is good.

As shown in Figure 13, when the second hole grouting project has been implemented for a total of 107 h, the critical grouting pressure is 8.5 MPa, and the cumulative injected cement grout volume is 1089 m\(^3\). The grouting pressure of the hole rises stepwise. According to the curve analysis, the grouting pressure of this hole changes slowly, which indicates that the injected rock mass has good permeability and a relatively large effective fracture width, but the fracture range is relatively small. In the process of cement grout diffusion, affected by factors such as slurry hardening and pressure splitting, the grouting pressure rises in a stepwise pattern, but generally, the fracture filling rate is high, and the cement grout near the grouting point diffuses fully.

Notably, in the grouting project of the Zhaoguan coal mine, an obvious pressure drop occurs after the grouting pressure reaches approximately 8.5 MPa. This shows that after reaching the grouting pressure, the fractures in the grouting area will be split into a large extent and connected with each other to form a new pressure channel, which makes the cement grout further diffuse, indicating that the theoretical value of the grouting pressure is reasonable.

5. DISCUSSION

5.1. Analysis of the Grouting Pressure Curve.

Combined with the observations of the actual grouting project in the Zhaoguan coal mine, a generalized model of the grouting pressure curve is established in this paper. The drilling pump pressure at the surface presents two forms. The first is that the grouting pressure gradually increases with the injection time until the critical grouting pressure is reached, which is called the “A” curve, as shown in Figure 14a. The second is that the grouting pressure changes in stages with time. In the initial stage of grouting, the grouting pressure rises slowly. Then, the grouting pressure rises rapidly, fluctuates, and changes within a certain range, and grouting finally ends after reaching the end pressure of grouting, which is called the “B” curve, as shown in Figure 14b.

The “A” grouting pressure curve suggests that the rock mass near the grouting point has poor permeability, reflecting a
small fracture space and local fracture blockage. Under the action of high-pressure grouting, it is possible to generate new fractures, but the conduction range is limited, the fracture spacing is small, and the grouting pressure rises slowly. The longer the time to reach the end pressure of grouting is, the larger the fracture range around the grouting point and the farther the grouting diffusion range. The “B” grouting pressure curve suggests that the injected rock mass has good permeability and a large effective fracture width. After the cement slurry is injected into a fracture, it takes a certain time to fill the fracture and produce the grouting pressure. After the grouting pressure is generated, the grouting pressure rises rapidly and continues to rise after a certain period of time. This situation indicates that the surrounding fractures are connected.

Figure 12. Ground grouting pressure curve of the first hole.

Figure 13. Ground grouting pressure curve of the second hole.

Figure 14. Generalized models of a drilling pump pressure curve: (a) “A” curve and (b) “B” curve.
6. CONCLUSIONS

(1) The conceptual model of fracture grouting by directional drilling is established. Considering the additional pressure such required to overcome the pressure loss and cement grout self-weight pressure caused by the movement of cement grout along the cement grout transportation pipeline and grouting hole, the relationship between the ground grouting pressure and the critical grouting pressure is determined: critical grouting pressure is equal to ground grouting pressure plus cement grout self-weight pressure.

(2) A fracture mechanics analysis model of a fractured rock mass is constructed. Based on H-B strength criterion, the critical head pressure of hydraulic fracturing is analyzed, and the theoretical calculation equation of the final pressure of fracture grouting based on the H-B strength criterion is obtained. The determination methods of the main parameters in the equation calculation, such as the uniaxial compressive strength and internal friction angle, the H-B constant of an intact rock block, rock mass disturbance parameters, GSI, and crustal stress, are determined.

(3) Combined with the above research and equation calculation, the theoretical value of the critical grouting pressure of the surface grouting project of the Xujiazhuan limestone aquifer of the Benxi Formation is 14.52 MPa, and the theoretical value of the ground grouting pressure is approximately 8.5 MPa.

(4) According to the engineering practice, the change in the ground grouting pressure in the actual grouting project of the Zhaoguan coal mine presents two forms. The first is that the grouting pressure increases gradually with the injection time until the critical grouting pressure is reached. The second is that the grouting pressure changes in stages with time. In the early stage of grouting, the grouting pressure rises slowly. Then, the grouting pressure rises rapidly, fluctuates, and changes within a certain range, finally reaching critical grouting pressure.

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W.Z. performed the methodology, formal analysis, and wrote the original draft of the manuscript. F.W. performed the validation and investigation. C.H. performed the project administration, funding acquisition, and reviewed and edited the manuscript. X.L. performed the data curation. Z.P. performed the visualization. Q.R. performed work in software and resources. F.Y. performed the formal analysis.

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Notes
The authors declare no competing financial interest.

■ REFERENCES

(1) Peng, S. P. Present Study and Development Trend of The deep coal resource distribution and mining geologic evaluation. Coal 2008, 2, 1–11.
(2) Lan, H.; Chen, D. K.; Mao, D. B. Current status of deep mining and disaster prevention in China. Coal Sci. Technol. 2016, 44, 39–46.
(3) Xing, H. A. Study on water bursting regular pattern at deep water-bearing strata. Chin. Mining. Mag. 2012, 21, 115–118.

(4) Wang, X. F. Grouting water plugging technology of underground mining for water-rich mine and the related case analysis. Metal Mine. 2014, 8, 129–133.

(5) Kan, X. D.; Tian, L.; An, X. L.; Guo, S. T.; Yang, X.; Heng, P. G. Application and development trend of grouting technology in prevention and control of Ordovician limestone water disaster in North China type coalfields. IOP Conf. Ser.: Mater. Sci. Eng. 2021, No. 012161.

(6) Wu, Q. Progresses and prospects of prevention and control technology of mine water and reutilization in China. J. Chin. Coal. Soc. 2014, 39, 795–805.

(7) Yao, N. P. Development trend of nearly horizontal directional drilling technology in coal mines of China. Coal. Sci. Technol. 2020, 38, 76–80.

(8) Yuan, H.; Deng, Y.; Pu, C. Y.; Zhang, D. Z.; Shao, G. Y.; Dong, X. W.; Yang, M.; Li, S. Z.; Yang, X.; Chen, J. Grouting technology of L type surface borehole applied to water disaster prevention in mine. Coal Sci. Technol. 2017, 45, 171–175.

(9) Zhao, Q. B. Technical countermeasures and guidance principles of seam safety mining above high pressurized water aquifer. Coal. Sci. Technol. 2013, 41, 83–86.

(10) Zhang, S. T.; Ma, H. W.; Ji, Y. D. Optimization of regional advanced coal floor water hazard prevention and control technology and its application. Coal. Geol. Explor. 2021, 49, 167–173.

(11) Zhao, Q. B.; Jiang, Q. M.; Gao, C. F. Study on floor water inrush mechanism of deep seam in Hanxing Mining Area. Coal Sci. Technol. 2016, 44, 117–121.

(12) Jiang, X. M.; Ren, H. J.; Chen, Y. Z. Application of advance exploration and control technology in deep mining of Hanxing Mining Area. Coal Eng. 2020, 52, 66–71.

(13) Zhao, Q. B. Ordovician limestone karst water disaster regional advanced governance technology study and application. J. China Coal. Soc. 2014, 39, 1112–1117.

(14) Zhao, Q. B. Technology of regional advance water prevention and control applied to pressurized coal mining zone above Ordovician limestone karst water. Coal. Sci. Technol. 2014, 42, 1–421.

(15) Zhao, Q. B.; Zhao, X. N.; Wu, Q. J.; Liu, C. W.; Wang, X. L. Water burst mechanism of “divided period and section burst” at deep coal seam floor in North China type coalfield mining area. J. Chin. Coal. Soc. 2015, 40, 1601–1607.

(16) Zhao, Q. B.; Bi, C.; Hu, W. Y.; Wang, H. Q.; Nan, S. H.; Liu, Z. B.; Chuai, X. Y. Study and application of three-stage seriflux diffusion mechanism in the fissure of aquifer with horizontal injection hole. J. Chin. Coal. Soc. 2016, 41, 1212–1218.

(17) Dong, S. N.; Wang, H.; Zhang, W. Z. Judgement criteria with utilization and grouting reconstruction of top Ordovician limestone and floor damage depth in North China coal field. J China Coal. Soc. 2019, 44, 2216–2226.

(18) Dong, S. N.; Guo, X. M.; Liu, Q. S.; Wang, H.; Nan, S. H.; Zheng, S. T.; Wang, Y. H. Model and selection criterion of zonal preact grouting to prevent mine water disasters of coal floor limestone aquifer in North China type coalfield. Coal Geol. Explor. 2020, 48, 1–10.

(19) Wu, S. Y.; Liu, L.; Chen, J.; Xu, Q. G. Research on precise grouting to prevent water disaster technology in Huanghebei Coalfield. Coal Sci. Technol. 2019, 47, 34–40.

(20) Zhang, D. Y.; Jiang, Q. M.; Gao, C. F.; Wang, T. J.; Wang, X. R. Study progress on key technologies for regional treatment of Karst water damage control in the floor of North China Coalfield. Coal Sci. Technol. 2020, 48, 31–36.

(21) Zheng, S. T. Application of complete set of surface bedding borehole exploration and grouting technology on floor high pressure karst water hazard governance. Coal Geol. Chin. 2018, 30, 53–57.

(22) Han, C. H.; Wei, J. C.; Zhang, W. J.; Yang, F.; Yin, H. Y.; Xie, D. L.; Xie, C. Quantitative permeation grouting in sand layer with consideration of grout properties and medium characteristics. Constr. Build. Mater. 2022, 327, 126947.