The deterministic description of rock excavation stability with a safety factor is frequently insufficient and, sometimes, misleading. The applications of the probabilistic analysis to determine the likelihood of a rock excavation failure have been proposed by many researchers. The probabilistic analysis will lead to a better understanding of excavation stability and provide more complete information on the safety and stability of rock excavations.

The difficulty in estimating the failure probability of a underground excavation is the complexity of mathematics involved. In order to overcome this limitation, the first-order second-moment approximation of the failure probabilities is utilized. The technique simplifies the computation significantly and allows for the analysis of relatively complex stability problems.

A few case studies are presented in the paper to illustrate the principles and applications of the probabilistic analysis of rock excavation stability, including specific discussion on the removable probability and failure probability of key blocks as defined in the block theory. The study indicates that the distributions of parameter values play a significant role in the excavation stability, which cannot be disclosed by the conventional methods of stability analysis. Greater dispersiveness of the parameter values will lead to a different probability of failure, although the factor of safety estimated by conventional methods remains the same if the mean values of the parameters are unchanged.

1 Introduction

The stability of rock excavations depends on many factors, such as mechanical properties of rock masses, orientations and properties of rock discontinuities, hydrological conditions, excavation geometries, and others. In an excavation stability analysis, these parameters are acquired by either in-situ investigations and/or laboratory tests. Because of the complex nature of rock masses, these parameters do not usually appear to be uniquely valued, but vary over a certain range. With the conventional limit equilibrium analysis, the average values of these parameters are used and a factor of safety, a deterministic number, is computed to indicate the stability of an excavation. This deterministic description of excavation stability, however, does not take into account of the uncertainties involved. It is, therefore, frequently insufficient or inaccurate and, sometimes, misleading. The uncertainty associated with rock excavation stability has been brought to the attention of some investigators (Tyler, et al, 1991; Hatzor and Goodman, 1992; Rethati, 1988; Piteau, 1977). Many have concluded that employing probabilistic analysis to determine the likelihood of an excavation failure will provide a better understanding of excavation stability and lead to more rigorous designs of rock excavations.

With a high degree of variations, the rock mass parameters for stability analysis can be regarded as random variables and characterized by various probability distribution functions. The excavation stability may, therefore, be considered as a random system, where the occurrence of a failure is a random event depending on the outcome of the random variables involved. Over the years, a number of studies have adopted this concept to determine the probability of excavation failure and to include the analysis of
uncertainty in rock excavation designs (Piteau, 1977; Moss and Steffen, 1978). Most of these studies, however, rely on Monte Carlo simulations to estimate the probability of rock excavation failure. A more rigorous analytical solution seems to be lacking. In order to provide a better insight into the probability of rock excavation failure, analytical studies based on probability theory is presented in this paper.

Furthermore, the block theory developed by Goodman and Shi (1985) has been successfully applied for rock excavation stability analysis. A critical step in the analysis is the determination of the key blocks – unstable removable blocks. In the original formulation of the block theory, however, the determination of the key blocks is based upon deterministic input data values, the most critical ones being the orientation of discontinuities and their strength properties. It is well accepted that the orientation of rock discontinuities in a rock mass follows certain random distributions. The average values of these random distributions are applied in the block theory for the identification of key blocks. Parametric analyses are frequently performed to compensate the limitation of the deterministic analysis. However, a better and possibly more accurate understanding of the rock excavation stability can be achieved if probabilistic analysis is conducted to determine the likelihood of a key block occurrence and its failure probability.

Based on the theory of probability, direct integrations can be utilized for the estimation of failure probabilities of excavations. Due to the complexity of mathematic operations, this approach can only be applied to solve relatively simple problems. In order to overcome the limitation of the direct integration method, the first-order second-moment approximation of the failure probabilities is adopted in this study. This approach allows for the analysis of more complex and more realistic excavation stability problems. Probabilistic analysis is also applied to evaluate the likelihood of a key block occurrence and its failure probability based on the given distribution of the discontinuities in the rock mass and the properties of the discontinuities with the first-order second-moment approximation. Case studies are presented in the paper to illustrate the proposed procedure for probabilistic analysis and to demonstrate the effects of random variable distributions on the likelihood of a key block occurrence and its failure probability.

2 The Failure Probability of Rock Excavations

If Q is the load applied to an excavation structure and R is the strength of the structure, the factor of safety

$$F_s = \frac{R}{Q}$$

of a rock excavation can be evaluated by the conventional limit equilibrium analysis with the following equation:

$$P_f = \int_{-\infty}^{\infty} [\int_{-\infty}^{+\infty} f_R(r)dr] f_Q(q)dq$$

Both R and Q are functions of a number of parameters which can be regarded as random variables. The resulting load Q and the strength R are, therefore, also randomly distributed. If the probability density functions (PDFs) of R and Q are fR(r) and fQ(q) having the shapes shown in Figure 1, failure will occur when the strength is less than the loading force, i.e. R < Q. The probability of this occurrence can be
evaluated by the two-fold integral below:

\[
\int_{-\infty}^{\infty} f_R(r) f_Q(q) \mathbb{I}(R-Q<0) \, dr \, dq \tag{1}
\]

where:

- \( Pr \) - probability of excavation failure
- \( R \) - strength of the excavation structure
- \( Q \) - load on the excavation structure
- \( f_R(r) \) - PDF of \( R \)
- \( f_Q(q) \) - PDF of \( Q \)

The probability density functions of \( f_R(r) \) and \( f_Q(q) \) for any realistic stability problems are usually very complex and a close-formed solution of the failure probability is very difficult, if not impossible, to obtain. In order to overcome this difficulty, the approximation technique is employed to estimate the failure probability as delineated below.

### 3 The First-Order Second-Moment Approximation of Failure Probability

For practical problems of rock excavation stability, the available information is frequently limited to the means and variances of parameters that are regarded as random variables, that is, the first and second moments of the random variables. Under this condition, the estimation of the failure probability is limited to a formulation based on the first and second moments of the random variables, i.e. the second-moment formulation (Ang and Tang, 1984).

\[
g = R - Q \tag{3}
\]

As given in Equation 2, the failure probability \( Pf = P(R<Q) = P(R-Q<0) \). Define performance function \( g \) as:

If \( g > 0 \), the rock excavation being studied is in the safe state, whereas if \( g < 0 \), it is in the failure state.

\[
g(x) = g(x_1, x_2, \ldots, x_n) \tag{4}
\]

Since \( R \) and \( Q \) are functions of a number of random variables, \( g \) is also a function of those random
variables, i.e.:

The equation \( g(x) = 0 \) defines the boundary between the region of safe state and that of failure state. When \( g(x) \) is a linear function, \( g(x) = 0 \) is a flat plane in a \( n \)-dimensional space. For \( n = 2 \), the plane is reduced to a straight line as shown in Figure 2.

![Figure 2 Performance Function](image)

The distance from \( g(x) = 0 \) line to the origin (Figure 2) is defined as the safety index \( \beta \), which can be estimated by the following equation (Ang and Tang, 1984):

\[
\beta = \frac{\mu_{x_1} - \mu_{x_2}}{\sqrt{\sigma_{x_1}^2 + \sigma_{x_2}^2}}
\]  

(5)

\[
P_f = 1 - \Phi(\beta)
\]  

(6)

The distance from \( g(x) = 0 \) line to the origin (Figure 2) is defined as the safety index \( \beta \), which can be estimated by the following equation (Ang and Tang, 1984):

If the random variables have normal distributions, the probability of failure can then be estimated by the standard normal distribution as below (Ang and Tang, 1984):

For nonnormal distributions, transformations can be performed to obtain equivalent normal distributions. The mean value and the standard deviation of the equivalent normal distribution for a random variable \( X_i \) can be estimated as:

\[
\mu_{X_i}^N = x_i^* - \sigma_{X_i}^N \Phi^{-1}[F_{X_i}(x_i^*)]
\]

\[
\sigma_{X_i}^N = \frac{\phi(\Phi^{-1}[F_{X_i}(x_i^*)])}{f_{x_i}(x_i^*)}
\]

(7)

where: \( \mu_{X_i}^N, \sigma_{X_i}^N \) - the mean value and standard deviation of the equivalent normal distribution for \( X_i \);

\( f_{X_i}(x_i^*), F_{X_i}(x_i^*) \) - the original PDF and CDF of the random variable \( X_i \) evaluated at \( x_i^* \); \( \phi(-), \Phi(-) \) - the PDF and CDF of the standard normal distribution.
If the performance function \( g(x) \) is nonlinear, the equation \( g(x) = 0 \) is a curved surface in the space. Figure 3 shows the curve of \( g(x) = 0 \) in a two-dimensional space. The curve can be either convex or concave as shown in the figure. To calculate the exact failure probability of \( g(x) < 0 \) for this nonlinear performance function, it generally requires complex integrations. For practical purposes, approximation to the exact probability will be necessary. According to Shinozuka (1983), the point \( (X^*) \) on the failure surface \( g(x) = 0 \) with the minimum distance to the origin is the most probable failure point. Taking a Taylor series of the function at \( (X^*) \) and truncating at the first-order term produce a tangent plane at \( (X^*) \). This tangent plane to the failure surface may then be used to approximate the actual failure surface and the safety index \( \beta \) may be evaluated as in the linear case. This first-order approximation of \( g(x) = 0 \) at \( X^* = (X_1^*, X_2^*, \ldots X_n^*) \) is expressed as (Ang and Tang, 1984):

\[
\sum_{i=1}^{n} (x_i - X_i^*) \left( \frac{\partial g}{\partial x_i} \right) = 0
\]

(8)

The most probable failure point is (Shinozuka, 1983):

\[
X_i^* = -\alpha_i^* \beta
\]

where \( \alpha_i^* \) is evaluated by:

\[
\alpha_i^* = \frac{\left( \frac{\partial g}{\partial x_i} \right)}{\sqrt{\sum_{i=1}^{n} \left( \frac{\partial g}{\partial x_i} \right)^2}}
\]

(9)

All the derivatives are evaluated at \( (X^*) \) and the solution of \( g(X^*) = 0 \) yields \( \beta \).

Figure 3 Nonlinear Function and Tangent Plane

(After Ang and Tang, 1984)

The value of \( \beta \) can be evaluated by going through several iterations with a simple numerical algorithm proposed by Rackwitz (1976) and the probability of failure is then estimated by Equation (6). Due to the linear approximation of the nonlinear function, the failure probability is either slightly overestimated or
underestimated depending on convexity of the function, but in most cases a sufficiently accurate approximation can be obtained. Detailed discussion on the accuracy of the linear approximation can be found elsewhere (Fiessler, et al, 1979). With this first-order second-moment approximation, probabilistic analyses for more complex and practical problems can be performed without much difficulty. By adopting this approach, FORTRAN90 programs are developed employing the algorithm proposed by Rackwitz and the failure probabilities of various cases of rock excavations can be determined.

4 Failure Probability of a Triangular Roof Prism

In this study, a simplified two-dimensional analysis is conducted to determine the failure probability of a triangular roof rock prism. Consider a long uniform asymmetric triangular prism formed in the roof of the excavation by inclined joints as shown in Figure 4. The prism is subject to horizontal stress \( \sigma_h \). The static equilibrium is maintained by the resistance \( P \), the support \( R \) and the weight \( W \). The resistant force and the load are calculated as force for per unit length. Following the relaxation analysis proposed by Brady and Brown (1993), the safety factor of the asymmetric triangular roof prism can be estimated as:

\[
F_s = \frac{P}{W - R}
\]

where \( P \) is the resistance preventing the prism from falling, which is due to the horizontal-stress-induced frictional force and the support. The resistance \( P \) per unit length can be calculated by the following equation:

\[
P = \frac{\lambda \gamma H h}{\sin \phi} \left[ \sin \alpha_1 \sin (\phi - \alpha_1) + \sin \alpha_2 \sin (\phi - \alpha_2) \right]
\]

and \( W \) is the weight of the prism per unit length which can be estimated as follows:

\[
W = \frac{\gamma h^2}{2} \left[ \tan \alpha_1 + \tan \alpha_2 \right]
\]

(10)

where:

- \( \alpha_1, \alpha_2 \) - prism angles as shown in Figure 4
- \( \phi \) - internal frictional angle of the discontinuities
- \( \lambda \) - in-situ horizontal to vertical stress ratio;
- \( \gamma \) - average overburden density
- \( H \) - depth of the excavation
- \( h \) - prism height
- \( R \) - support force per unit length
Figure 4 Cross-sectional View of an Asymmetric Triangular Roof Prism

In the analysis, $\alpha_1$, $\alpha_2$, $\phi$, $\lambda$, $h$, and $R$ are considered to be random variables and the performance function for the FOSM approximation is defined as:

$$
g = \frac{\lambda h H}{\sin \phi} \left[ \sin \alpha_1 \sin (\phi - \alpha_1) + \sin \alpha_2 \sin (\phi - \alpha_2) \right] - \frac{\gamma h^2}{2} (\tan \alpha_1 + \tan \alpha_2) + R \tag{11}$$

Although the function is in a relatively complex form, the probabilistic analysis can still be conducted without much difficulty. With the depth of the excavation $H = 122$ m and the random variable distributions given as:

- $\alpha_1$ : mean $\mu_{\alpha_1} = 45^\circ$, standard deviation $\sigma_{\alpha_1} = 0.1 \mu_{\alpha_1}$
- $\alpha_2$ : mean $\mu_{\alpha_2} = 35^\circ$, standard deviation $\sigma_{\alpha_2} = 0.1 \mu_{\alpha_2}$
- $\phi$ : mean $\mu_{\phi} = 35^\circ$, standard deviation $\sigma_{\phi} = 0.1 \mu_{\phi}$
- $\lambda$ : mean $\mu_{\lambda} = 0.3$, standard deviation $\sigma_{\lambda} = 0.1 \mu_{\lambda}$
- $h$ : mean $\mu_h = 4.5$ m, standard deviation $\sigma_h = 0.1 \mu_h$
- $R$ : mean $\mu_R = 950$ kN/m, Standard deviation $\sigma_R = 0.1 \mu_R$

determine the probability of prism failure is estimated to be $P_f = 0.7$. The above parameters are also varied over certain ranges to investigate the effects of various parameters upon the failure probability. The corresponding safety factors are also calculated, as presented below.

The effects of individually changing the mean of the horizontal to vertical stress ratio $\lambda$ and the mean of the frictional angle $\phi$ while keeping all other parameters constant are illustrated in Figures 5 and 6, along with the corresponding safety factor computed by the conventional method.
As indicated, the probability of failure varies nonlinearly as the means change. Above a certain value of the safety factor, the probability of failure approaches asymptotically to zero. Any further increase of factor of safety will have little effect on the failure probability of the roof prism. However, the failure probability has different degree of sensitivity to the changes of different variables. For the frictional angle $\phi$, the mean value needs to be increased for the safety factor to reach 4.0 in order to reduce the failure probability to 0.1. As for the horizontal to vertical stress ratio $\lambda$, the mean value needs only to be increased for the safety factor to reach 1.6 in order to achieve the same level of failure probability. At the same safety factor level of 1.6, it may mean completely different failure probability under different conditions. It shows that the safety factor may not provide complete information regarding the stability of the roof prism.

The ratios of standard deviation over mean are also varied over a range while keeping the mean values constant. The results reveal that if the initial failure probability is greater than 0.5, the failure probability will decrease as the standard deviation increases and if the initial failure probability is less than 0.5, the failure probability will increase as the standard deviation increases. This applies to all random variable parameters. A typical curve is illustrated in Figure 7, where the standard deviation of the frictional angle $\phi$ is varied over a range (expressed as the ratio of $\sigma_\phi/\mu_\phi$), while the other parameters remain constant.
5 Failure Probability of a Key Block

The block theory developed by Goodman and Shi (1985) has been successfully applied for rock excavation stability analysis. A critical step in the analysis is the determination of the key blocks – unstable removable blocks. In the original formulation of the block theory, however, the determination of the key blocks is based upon deterministic input data values, the most critical ones being the orientation of discontinuities and their strength properties. It is well accepted that the orientation of rock discontinuities in a rock mass follows certain random distributions. The average values of these random distributions are applied in the block theory for the identification of key blocks. Parametric analyses are frequently performed to compensate the limitation of the deterministic analysis. However, a better and possibly more accurate understanding of the rock excavation stability can be achieved if probabilistic analysis is conducted to determine the likelihood of a key block occurrence and its failure probability.

In this study, analysis is conducted to identify random distribution patterns of rock discontinuity parameters. Probabilistic analysis is applied to evaluate the likelihood of a key block occurrence and its failure probability based on the given distribution of the discontinuities in the rock mass and the properties of the discontinuities.

Removable Blocks and Removable Probability

Based on the block theory, a key block is a finite removable block that is unstable without support. In this paper, to estimate the occurrence probability of a key block, a two-step approach is taken. First, the probability of a rock block becomes removable is determined and based on the strength properties of the discontinuities forming the block, the block failure probability is then estimated.

According to the block theory, the necessary and sufficient condition for a block to be removable is:

\[ \text{JP} \neq \emptyset \]

\[ \text{BP} = \emptyset \]

where JP is the joint pyramid and BP is the block pyramid, which is the intersection of the joint pyramid, JP, and the excavation pyramid, EP, i.e.:

\[ \text{BP} = \text{JP} \cap \text{EP} \]
An equivalent statement of the above condition is that for a block to be removable, the block formed by joint planes alone is infinite and the block formed by joint planes and free surfaces together is finite.

Figure 15 displays the stereographic projection of 4 discontinuity planes and an excavation plane, each represented by a circle on the projection. These discontinuity planes are actual data collected from a field study with known distributions. The mean values are used for the stereographic projection in the figure. The joint pyramids, i.e. JPs, are identified with associated codes. The excavation space is below the excavation plane, representing a mine stope under the hanging wall, and the rock mass is above the plane. Therefore, EP on the projection is the area inside of the circle of the excavation plane. Based on the block theory, the JPs that fall completely outside the circle of the excavation plane are removable blocks and those removable blocks that will slide out of the rock mass without support are key blocks.

Assuming the joint planes are randomly distributed in the rock mass, the JPs will have certain probability to fall either in- or outside the circle of the excavation plane, partially or completely. The problem of the occurrence probability of a removable block is now converted to one that determines the probability of the block’s JP completely falls outside of the circle. As shown in Figure 8, for a JP to extend into the excavation circle, some corners are the first points to reach into the circle. The probability of a block being removable can, therefore, be further simplified to one that determines the probability of the JP’s corners fall outside of the excavation circle. There are a number of corners for any given JP. In many cases, there will be one corner that is most likely to extend into the circle, which often dominates the forming of a removable block. In this paper, it is assumed that only one corner has a relatively high probability to extend into the circle of the excavation plane. The other corners of the JP are all outside of the circle and have much lower probability extending into the circle and their significance on the forming of a removable block can be neglected. The focus of this study is, therefore, on the probability of one particular corner being outside of the circle of the excavation plane and making the block removable. The probability of more than one corner being outside of the circle requires further analysis and will be dealt with in the next step of the study.

A corner point of a JP on the stereographic projection represents an edge of a rock block in the space. Whether or not a corner point falls outside of a circle on the projection indicates if or not the edge is below the plane represented by the circle. Any edge of a rock block is the intersection line of two joint planes. In the analysis of the removability of a rock block as presented in the block theory, only the orientations of
joint planes need to be considered and, therefore, all joints are modeled with planes passing through the origin \((0, 0, 0)\). A pair of joint planes passing through the origin can be defined as:

\[
A_1 X + B_1 Y + C_1 Z = 0 \\
A_2 X + B_2 Y + C_2 Z = 0
\]

Where \(A_1, A_2, B_1, B_2, C_1\) and \(C_2\) are parameters defined by the dip angles and the dip directions of the joint planes. They can be expressed as:

\[
A_1 = \sin \alpha_1 \sin \beta_1; \quad B_1 = \sin \alpha_1 \cos \beta_1; \quad C_1 = \cos \alpha_1 \\
A_2 = \sin \alpha_2 \sin \beta_2; \quad B_2 = \sin \alpha_2 \cos \beta_2; \quad C_2 = \cos \alpha_2
\]

where \(\alpha_1\) and \(\alpha_2\) are the dip angles for the joint planes 1 and 2; and \(\beta_1\) and \(\beta_2\) are the dip directions for the two planes respectively. Since the dip angles and the dip directions are random variables, the six parameters given by these angles are also random variables that define two randomly distributed planes in the rock mass.

The intersection line of these two planes is a line passing through the origin \((0, 0, 0)\) and can be defined as:

\[
X = pt ; \quad Y = qt ; \quad Z = rt
\]

(12)

where \(p, q\) and \(r\) are direction numbers defining the orientation of the line. The variable \(t\) is a reference variable and varies in the range of \(0 < t < \infty\) or \(-\infty < t < 0\), depending on which side of the half spaces of a plane the line is pointing to. The direction numbers of the line, \(p, q\) and \(r\), are random variables defined by the parameters of the two random joint planes and are given as:

\[
p = \sin \alpha_1 \cos \beta_1 \cos \alpha_2 - \cos \alpha_1 \sin \alpha_2 \cos \beta_2 \\
q = \cos \alpha_1 \sin \alpha_2 \sin \beta_2 - \sin \alpha_1 \sin \beta_1 \cos \alpha_2 \\
r = \sin \alpha_1 \sin \beta_1 \sin \alpha_2 \cos \beta_2 - \sin \alpha_1 \cos \beta_1 \sin \alpha_2 \sin \beta_2
\]

The excavation plane can be expressed as (The plane is made to pass through the origin to facilitate the block removability analysis.):

\[
AX + BY + CZ = 0
\]

(13)

The excavation plane is considered a fixed plane and the plane parameters \(A, B\) and \(C\) are deterministic numbers. They are defined by the dip angle and the dip direction of the plane as given below:

\[
A = \sin \alpha \sin \beta; \quad B = \sin \alpha \cos \beta; \quad C = \cos \alpha
\]

where \(\alpha\) and \(\beta\) are the dip angle and the dip direction of the plane respectively. With the excavation plane defined, whether or not the joint block with the edge defined by Equation 12 is removable depends on whether or not the edge is below the excavation plane. This is indicated on the stereographic projection with the JP’s corner falls outside of the circle representing the excavation plane. Therefore, the probability that the block is removable can be defined as:

\[
P[\text{block removable}] = P[A(pt) + B(qt) + C(rt) < 0]
\]

(14)

Here it is assumed that the reference number \(t\) may take any value in the range of \(0 < t < \infty\). It can be shown without difficulty that for a given value of \(t\), e.g. \(t = t_0\), if:

\[
A(pt_0) + B(qt_0) + C(rt_0) < 0
\]

then:

\[
A(pt) + B(qt) + C(rt) < 0
\]
for all the possible values of \( t \). This applies to the reversed inequality as well. Based on the above analysis, the reference number \( t \) is, therefore, assigned a value of 1 to facilitate the mathematic manipulation. The probability that the block is removable is then expressed as:

\[
P[\text{block removable}] = P[A p+B q+C r<0]
\]

(15)

Since the parameters \( p, q \) and \( r \) are all nonlinear functions of multi random variables, \( \alpha_1, \alpha_2, \beta_1 \) and \( \beta_2 \), applying direct integration to compute the probability is formidable. The First Order Second Moment (FOSM) approach is, therefore, employed.

A case study is conducted here for the removable probability of a rock block, employing RBK.F90, a short FORTRAN90 program specifically developed for this study. The set of data that were acquired from an actual field study as illustrated in Figure 15 is used for this case study. The numerical values of the data set are presented in Table 1. The beta distribution is applied to all the dip angles and dip directions. Table 1 also includes the estimated distribution parameters. The excavation surface represents a hanging wall (HW) in an underground stope with a dip angle of 15° and dip direction of 40°. The excavation surface is considered a fixed plane and the orientation parameters are deterministic values. The excavation space is below the surface and the jointed rock mass is above the surface.

| Orientation Parameters for Joint Planes and Hanging Wall |
|---|---|---|---|---|
| dip/dip direction | Beta Distribution Parameters (dip/dip direction) |
|  | \( a \) | \( B \) | \( q \) | \( r \) |
| J1 | 41°/121° | 27°/86° | 69°/149° | 3.0/9.0 | 7.0/7.0 |
| J2 | 75°/231° | 52°/212° | 92°/246° | 6.0/7.0 | 4.0/5.0 |
| J3 | 53°/283° | 40°/275° | 74°/292° | 6.0/8.0 | 9.0/11.0 |
| J4 | 57°/318° | 46°/298° | 75°/332° | 9.0/13.0 | 14.0/9.0 |
| HW | 15°/40° | n/a | n/a | n/a | n/a |

As shown in Figure 15, two blocks, i.e. Block 1001 and Block 1111, are of interest for removable probability study. They both have a relatively even chance to be either removable or non-removable (tapered) and are, therefore, the focus of the study. The other blocks identified in the stereographic plot are either almost certain removable or almost certain non-removable and are not included in the following analyses.

As shown in Figure 15, the intersection line of Joints 1 and 4 forms an edge of Block 1111 in the upper side of the hanging wall (circle with dashed line), as indicated by I and a star in the stereographic plot. The intersection line also forms an edge of Block 1001 in the lower side of the hanging wall, as indicated by I’ and a star. These two points representing the upper and lower halves of the intersection line are relatively close to the circle representing the hanging wall. When point I moves out of the HW circle, Block 1111 becomes removable. The same applies to I’ for Block 1001. It can also be noticed that the probability of I moving into the HW circle is complementary to the probability of I’ moving into the same circle, which indicates that the removable probability of Block 1111 is complementary to the removable probability of Block 1001, i.e.:

\[
P(\text{Block 1111 removable}) = 1 - P(\text{Block 1001 removable})
\]

(16)
Running the program RBK.F90 with the parameters provided in Table 1 produces 0.021 for the removable probability of Block 1111. Based on Equation 16, the removable probability of Block 1001 is, therefore, 0.979.

Parametric study is also conducted by running the program RBK.F90 for various scenarios, including change of HW plane orientations and change of the degree of random variable dispersions. When reducing the dip angle of the HW plane, assuming it can be precisely controlled by mining excavation, the removable probability of Block 1111 increases while, in the same time, the removable probability of Block 1001 reduces. Figure 9 shows the variation of removable probability for Block 1111 over a range of the HW dip angles.

It is interesting to note that with an approximately 7° dip angle for the HW, the removable probability of Block 1111 is 0.5. With this orientation of the HW, the intersection line of Joints 1 and 4, represented by the point I on the stereographic plot, is on the circle representing the HW. At this point, Block 1111 has 50/50 chance to become removable. When the HW plane is almost horizontal, Block 1111 has a high probability to become removable. If other information indicates that Block 1111 may potentially cause large volume of roof caving, the result of this analysis suggests that the HW should be excavated with a greater dip angle whenever possible.

When changing the dip direction of the HW plane while keeping its dip angle at a fixed value, the removable probability changes too. Figure 10 shows the removable probability of Block 1111 as a function of HW dip direction with the dip angles fixed at 10° and 15° respectively.

The result indicates that the HW plane should be developed to dip more towards east whenever possible in order to avoid making Block 1111 removable. The key advantage of the probabilistic analysis is that it takes consideration of uncertainties in the input data, which cannot be disclosed by the deterministic analysis. In order to investigate the impact of the dispersion of the input data on the removable probability of rock blocks, the ratio of variance over mean for the input data is varied over a range while maintaining the mean value constant. Since the deterministic analysis uses only the mean value, the change of variance makes no difference. It illustrates in Figures 11 and 12, however, the removable probability of rock blocks changes considerably when the variance of the input data changes.
Figure 10 Removable Probability of Block 1111 with Different Hanging Wall Dip Direction

Figure 11 Removable Probability of Block 1111 with Different Variance in Input Data

Figure 18 shows removable probability of Block 1111 as the ratio of variance/mean varies from 0.1 to 3.5 while maintaining constant mean values of the input data. When the variance/mean ratios of the Joint 1 dip and dip direction increase from 0.1 to 3.5, the removable probability of Block 1111 increases from 0.055 to 0.36. While the variance/mean ratios for both Joints 1 and 4 dip and dip direction increase from 0.1 to 3.5, the probability increases from 0.129 to 0.433. These results indicate that the dispersion of input data plays an important role in determining the removable probability. The curve in Figure 18 also reveals that as the data dispersion reaches a certain level, an approximate variance/mean ratio of 3.0 in this case, the removable probability seems to approach asymptotic values, which may be considered as the limiting removable probability when the data are completely random. Figure 19 shows the removable probability of Block 1001 under the same conditions, which is the complement of the probability in Figure 18. Notice that the removable probability of Block 1001 decreases from high values as the variance/mean ratio increases and also approaches asymptotic values.
Probabilistic Analysis of a Mine Ventilation Shaft Stability

A vertical ventilation shaft was developed in an underground mine. The shaft, excavated by a raise borer, is approximately 90 m in length and 2.13 m in diameter as shown Figure 13. The surrounding rock is granite with several sets of joints. The discontinuities in the rock mass had major impacts on the stability of the shaft excavation. Initial static limiting-equilibrium key block analysis indicated that the shaft should be stable without support. In-depth probabilistic analysis, however, showed that due to random variations in both joint orientations and rock properties, there was a high probability of key block failure with significant rock volumes. The block theory developed is applied with the consideration of uncertainties in rock discontinuities and rock mass properties. Probabilistic analysis is applied to evaluate the likelihood of a key block occurrence and its failure probability based on the given distribution of the discontinuity orientations and the mechanical properties of the discontinuities.

Mine site investigation revealed that there are 3 major sets of joints that may have significant impact to the mine shaft stability. The variations in joint orientations fit the Beta distribution given by Equation 17 with the parameters defined in Table 2.

$$f_X(x) = \frac{1}{B(q,r)} \frac{(x-a)^{q-1}}{(b-a)^{q-1}} \frac{(b-x)^{r-1}}{(b-a)^{r-1}} \quad a \leq x \leq b$$

(17)
Table 2 Orientation Distribution Parameters for Joint Planes

<table>
<thead>
<tr>
<th>Mean (dip/dip direction)</th>
<th>Beta Distribution Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A</td>
</tr>
<tr>
<td>J1 73°/116°</td>
<td>66°/109°</td>
</tr>
<tr>
<td>J2 85°/272°</td>
<td>78°/265°</td>
</tr>
<tr>
<td>J3 30°/285°</td>
<td>23°/278°</td>
</tr>
<tr>
<td>φ 32°</td>
<td>0°</td>
</tr>
</tbody>
</table>

Note: φ - angle of friction for all the surfaces

The mean orientations of the joint sets are plotted on a stereographic net as shown in Figure 21. As can be seen on the plot (without considering those less likely repeated blocks), there are 8 block types, namely, 000, 001, 010, 011, 100, 101, 110, and 111. Since the ventilation shaft is vertical, the vector of the upward shaft axis is at the center of the reference circle (dashed line) and the vector of the downward shaft axis is infinitely away outside the reference circle, which can not be plotted. Based on the block theory, block types 000 and 111 are infinite blocks and the other block types: 001, 010, 011, 100, 101 and 110 are potential key blocks. Preliminary analysis indicated that, block types 010, 011, 100 and 101 would have minimal volume of caving and should not be of any major concern for the shaft stability. The analysis was, therefore, focused on the key block types 001 and 110, which would have relatively large volume of rock failure as illustrated in Figure 14.

In order for block 001 or 110 to become a key block, each block should have a downward sliding direction. As shown in Figure 15, the sliding direction for Block 001 is along the intersection line of J1 and J2 (I12). It is projected outside of the reference circle indicating a downward vector. For Block 110, it is along the intersection line of J2 and J3 (I23), also projected outside of the reference circle to point downward. Assuming the joint plane orientations are random variables, I12 and I23 may have a certain probabilities to fall within the reference circle and become upward vectors and Blocks 001 and 110 would, therefore, become stable blocks. The probability of this occurs is discussed below.
Running the FORTRAN90 program, RBK.F90, as discussed in the previous section, with the parameters provided in Table 2, produces very low probability for either block 001 or block 110 (0.0 and 0.00001 respectively) to be stable non-key blocks. Both blocks, therefore, are treated as potential key blocks with certainty.

The potential key blocks of 001 and 110 are approximately tetrahedrons formed by three joint planes, namely, J1, J2, J3 and the shaft excavation surface. As can be determined on the stereographic plot in Figure 21, both blocks may potentially slide on two joint planes. Block 001 may slide on J1 and J2, and Block 110 may slide on J2 and J3. Based on the block theory (Goodman and Shi, 1985) for double face sliding, the sliding force, $T$, can be calculated by:

$$T = \frac{\mathbf{w} \cdot (\mathbf{n}_i \times \mathbf{n}_j)}{\mathbf{n}_i \times \mathbf{n}_j}$$

(18)

and the resistance force, $R$, due to friction is given by:

$$R = N_i \tan \phi_i + N_j \tan \phi_j$$

(19)

where: $\mathbf{w}$ is the resultant force vector acting on the rock block (weight only in this study), $\mathbf{n}_i$ and $\mathbf{n}_j$ are the normal vectors of joint planes i and j respectively, $N_i$ and $N_j$ are normal forces on planes i and j respectively, and $\phi_i$ and $\phi_j$ are frictional angles for planes i and j respectively.

Considering only the dry friction of the rock blocks for resistance, the safety factors can, then, be calculated by:

$$S_F = \frac{R}{T}$$

(20)

Using the mean values in Table 2, the safety factors were calculated for the two concerned blocks of 001 and 110 to be:

$$S_{F(001)} = 1.1 \quad \text{and} \quad S_{F(110)} = 6.8$$

Both blocks seemed stable. However, all the parameters listed above, including the frictional angles are either a random variable itself or a function of several random variables. The failure probability of a key
block is, therefore:

\[ P_f = P(T - R > 0) \]  \hspace{1cm} (21)

Employing the FOSM procedure discussed in the last section, the performance function can be defined as:

\[ g = T - R = \frac{\mathbf{w} \cdot (\mathbf{n}_i \times \mathbf{n}_j)}{\mathbf{n}_i \times \mathbf{n}_j} - N_i \tan \phi_i - N_j \tan \phi_j \]  \hspace{1cm} (22)

Running the FORTRAN90 program, RBK.F90 as discussed in the previous section, the failure probabilities for the two blocks are estimated to be:

\[ P_{f(001)} = 0.485 \quad \text{and} \quad P_{f(110)} = 0.0012 \]

The analysis indicates that block 110 was of much less concern in terms of its stability. It had a very high safety factor of 6.8 and very low failure probability of 0.0012, and should be stable without support. The most critical block, however, was block 001. Although the safety factor is above 1.0, further probabilistic analysis showed a significant failure potential of 48.5%. To further verify the probabilistic analysis, Monte Carlo simulations are performed using the platform of “R”, a popular statistic analysis package, to check the failure probabilities and examine the distribution of safety factors. With several runs (100,000 simulations each), the Monte Carlo simulation produced similar failure probabilities as given by the program RBK.F90:

\[ P_{f(001)} = 0.475 \sim 0.49 \quad \text{and} \quad P_{f(110)} = 0.001 \sim 0.0013 \]

The distribution of safety factors generated by the simulation is also shown in Figures 16a and 16b. The results clearly indicated that for Block 001, there is a significant chance for the safety factor to drop below 1.0, while Block 110 is basically stable with much higher safety factor.
Another critical data in excavation stability analysis is the potential volume of rock block failure. Following the procedure proposed by Goodman and Shi (1985), it was identified that the potential maximum key block for both 001 and 110 would be formed by limiting planes along the projection lines of edges I13 and I23. The maximum key block can approximately be modeled as a tetrahedron, formed by the three joint planes, J1, J2, J3 and the excavation surface, approximated to be a flat cut surface. Since J1, J2, J3 and the limiting edges I13 and I23 are all functions of random variables of the joint orientations, the maximum potential rock failure volume also follows a random distribution. Following Goodman and Shi’s procedure and applying the random variable parameters listed in Table 1, the probabilistic distribution of potential rock failure volumes was estimated with the Monte Carlo simulation and was illustrated in Figure 17. The results revealed that the key block had a potentially sizable volume, most likely in the range of 8 to 13 cubic meters. This is a fairly significant volume and special attention has to be paid to the support and stability control of the excavation. Without proper control and support of the potential key blocks, the sliding of the blocks may cause severe damage and/or serious personal injury.

6 Concluding Remarks

Probabilistic analysis of rock excavation stability is performed in this study based on the theory of
probability. Due to the complexity of direct integration, the first-order second-moment approximation of the failure probability is adopted to solve complex and more practical excavation stability problems.

The parametric study indicates that as the mean values of the parameters change, both the probability of failure and the factor of safety change accordingly. However, the probability of failure is not linearly related to the factor of safety. Above certain level of the safety factor, the probability of failure approaches asymptotically to zero, meaning any further increase in safety factor makes little improvement on the excavation stability. In addition, the same level of safety factor may indicates completely different failure probabilities under different conditions, which demonstrates that the safety factor does not always provide complete information regarding the excavation stability. It is also observed that when the factor of safety equals 1.0, the corresponding probability of failure is approximately 0.5. This observation suggests that with a safety factor of 1.0, there should be a 50-50 chance for failure to occur.

A further observation of the parametric study indicates that with constant mean values but different standard deviations of the parameters, the factor of safety remains the same, but the probability of failure differs. A more scattered pattern of parameter values will lead to a higher degree of uncertainty. There will be less assurance in determining either a stable or an unstable excavation. To ensure the same degree of the excavation stability, different safety factors must be chosen depending on the distributions of those parameters that are regarded as random variables. Conventional methods select a factor of safety for an excavation regardless the dispersiveness of the parameter values involved, which often times leads to improper designs. The probabilistic analysis is able to take this uncertainty into consideration and provides more complete information.

Reference


CASE STUDY OF AN OPEN PIT COAL MINE DESIGN USING PIT OPTIMIZATION

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1 Introduction

Recently, the investment in overseas mineral resources development of Korea is increasing not only through the acquiring of equity stakes but also by direct mine operation. The Korea Coal Corporation is the first Korean company to set up operations in the Mongolian resource market by way of a direct operation. Based on a 60-year track record and know-how in the coal-mining industry in Korea, the Korea Coal Corporation has developed and operated the Nuurst Khotgor coal mine, which is a bituminous coal mine in Uvs Aimag in Mongolia. The Nuurst Khotgor coal mine, of which the average width of the coal seam is about 58m, the length of the coal seam is 3.7km and the overall dip angle of the coal seam is about 40 degrees, was planned to be developed as an open pit mine.

The Nuurst Khotgor coal mine was designed using a pit optimization method. A pit optimization method is a type of computer simulation technique considering the three-dimensional shapes of the ore body, the rock mechanical properties of the ore and the rock, and the economic conditions. In this case study, we provide the practical considerations for an open pit mine operation by reviewing the process of the mine design of the Nuurst Khotgor coal mine.

Keywords: pit optimization, schedule optimization, 3-D modeling, Nuurst Khotgor coal mine

2 Overview of Pit Optimization Method

The first step of pit optimization is the reconstruction of a 3-D block model based on input parameters which consist of three factors: the geographic and geological conditions of the ore body, the properties of the rock mass, and the economic conditions. The geographic and geological conditions of the ore body include such features as the boundary of the license area, the shape of the ore body, and the spatial distribution of the grade. The properties of the rock mass are used to define the specifications of the slope, such as the slope angle and the safety berm width, based on the mechanical properties of the rock and the properties of discontinuity. The economic conditions, such as the annual production rate, the coal price, and the mining cost, are also key factors to consider when optimizing an open pit design. The optimization algorithm is applied in order to simulate and calculate the value of the mined ore by sequential excavation using the 3-D block model. The optimal pit shell is selected from among various pit shells. It must correspond to the maximum value in an analysis of the calculated value of each pit shell. Schedule optimization was applied in this case to establish the optimized production plan according to the production capability, processing capability and spatial distribution of the grade corresponding to the selected optimal pit area from the pit optimization step.
3 Application of Pit Optimization in the Nuurst Khotgor Coal Mine

The Nuurst Khotgor coal mine is a sub-bituminous coal mine of which the length is 3.7km in the north-south direction. On the whole, the elevation of the northern part is higher than that of the southern part. The hanging wall of the coal seam consists of sandstone (partially imbedded with mudstone), and the foot wall consists of conglomerate materials. To apply the pit optimization scheme, a 3-D geological model and a block model were formulated using Gemcom GEMS, professional mine design software, based on a digital elevation map and a digitalized geological map. The block model is a three-dimensional matrix of rectangular blocks. The size of each block was determined to be 15m x 30m x 10m considering the geological condition and the designed bench height for excavation (refer to Figure 1). The specifications of the slope were determined considering the properties of discontinuity, such as joints, faults and bedding, the rock classification results by RMR, and the mechanical properties of the rock according to a laboratory rock test. The slope angle of the foot wall was determined to be equal to that of the dip angle of the coal seam, and the overall slope angle of the hanging wall was determined to be 52 degrees considering the stability of slope (refer to Figure 2). However, economic parameters such as the coal price, mining cost, washing cost, general and administrative costs, and marketing costs, among others, for pit optimization were applied based on the result of a feasibility study. Figure 3 (a) shows the increase in the tonnage of each pit shell and the corresponding increase in the mine value as calculated by Gemcom Whittle, which is professional mine optimization software. Figure 3 (b) shows the three-dimensional shape of the optimal pit, which is the seventh pit shell corresponding to the maximum NPV (Net Present Value). After selecting the optimal pit, the Miwawa algorithm® was applied to determine the annual production plan under the allowance of the production capability for each year (refer to Figure 4 (a)). The result of this schedule optimization process provided the pit outline for each year. Hence, a mine design accounting for the ramp and bench design was formulated while also considering the specifications of the mining equipment, the location of the processing plant, the location of the waste dump, and other factors. The designed pit layout is shown in Figure 4 (b).
Figure 2 Specifications of the slope applied to pit optimization

Figure 3 The result of pit optimization

Figure 4 The result of schedule optimization

4 Conclusions

The pit layout of the Nuurst Khotgor coal mine was designed as mentioned above. It is under operation following the detailed mine operation plan including the mine equipment, loading and hauling operations, drilling operations, blasting operations, manpower requirements, and processing and handling operations. Also considered are the location and size of the waste dump, the incidental facilities and other factors.

This article shows an example of an open pit design using 3-D geology and a deposit modeling
technique including a geo-statistical method and a pit optimization technique based on a computer simulation method. This type of technique is applied in many parts of the world, not only at the evaluation and design stage but also at the operating stage for the efficient management of production.

To operate an overseas mine directly, the demand for technology and know-how for mine design and operation will increase in keeping with economic evaluations. Particularly, technology and know-how for open pit mines must be accumulated because most of the mine sites in coal industry use the open-pit type.

Reference
POROSITY DEPENDENCY ON MECHANICAL PROPERTIES AND PORE-WATER PRESSURE RESPONSE OF SANDSTONE IN TRIAXIAL COMPRESSION

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Undrained triaxial tests performed on sandstone specimens with two different porosities showed that the observed induced pore-water pressure response is quicker in high porous sandstone and the maximum observed induced pore-water pressure is higher compared to that of low porous sandstone. Volumetric strain results showed that the dilation is a significant mechanism only at low confining pressures for both types of sandstone. Within the tested confining pressure range, a considerable difference in the onset of dilatancy relative to the peak differential axial stress was not observed.

1 Introduction

The knowledge of the mechanical properties of different rocks is vital for understanding a wide range of geo-mechanical problems. Laboratory tests are usually performed to derive many of those mechanical properties. Triaxial testing is a popular testing method for obtaining various mechanical properties such as stress-strain behaviour, volumetric strain behaviour, pore-pressure response, permeability changes… etc., under different stress conditions.

Rock mechanical behaviour is dependent on characteristics of the constituents of the material and in-situ environmental conditions. It has been repeatedly shown in the literature that mineralogy, density, porosity, grain size and water content [1-5] of a rock can have decisive impacts on its mechanical behaviour. Environmental factors such as strain rate, pressure and temperature are also of critical importance in governing the strength and deformational behaviour of rock [6-8]. In addition to the parameters mentioned above, rock mass behaviour is highly influenced by its inherent discontinuities and their mechanical and geometrical properties. However, the influence of discontinuities is beyond the scope of this paper as we consider only intact rock.

In spite of the extensive experimental and numerical investigations on the topic, still further work is required to obtain a more detailed characterization of the influence of material and environmental properties on the mechanical response of rock materials. In particular, the pore-pressure response (i.e. the induced pore-pressure variation against deformation) of saturated rocks with different porosities under different stress conditions is not thoroughly understood. The aim of this paper is to experimentally characterize the behaviour of saturated sandstone with two different porosities under high and low confining pressure conditions.
2 Porosity of rock and its implications for rock failure

Failure of a rock can be discussed as two end members (1) brittle failure (i.e. the rock sample shows considerable dilatancy and failure involving strain softening and brittle fracturing) under low confining pressures and (2) ductile failure (i.e. the rock sample shows diffuse creep and failure involving strain hardening) under high confining pressures. The failure mode under confining pressures in between those that produce brittle and ductile behaviour is complex and highly material dependent. Klein et al. [8] suggested that within the intermediate pressure range failure mode cannot be unambiguously classified as ‘brittle’ or ‘ductile’ for Bentheim sandstone.

The influence of rock porosity on mechanical behaviour is mainly associated with the nature of volume changes occurring during deformation in compression. Brace [6] reported that the volume change can take place due to three phenomena- elastic change in the mineral volume, compaction (reducing pore volume) and dilatancy (increasing pore volume). Generally, in a brittle regime, during the early stage of deformation, volume decreases due to compaction (by closure of pre-existing micro cracks) and elastic deformation of grain contacts. Conversely, close to the peak stress, volume significantly increases due to the inception of dilatancy. Once a material has transitioned to the fully plastic phase from the brittle phase, under elevated confining pressures, dilatancy disappears and the yield stress is completely independent of the confining stress. Under those circumstances the material undergoes a shear-enhanced compaction phase [8]. The behaviour of volumetric strain with mean stress for Bentheim sandstone under different confining stresses is presented in Klein et al. [8] and shown in Figure 1. As Figure 1 depicts, dilation is significant only at low confining pressures (Figure 1a) and it is absent at very high confining pressures.

![Figure 1: Mean stress versus volumetric strain for Bentheim sandstone at (a) low confining pressures and (b) elevated confining pressures. C* in Figure 1(a) represents the onset of dilation and in Figure 1(b) represents the onset of shear-enhanced compaction (Figure 2b and 3b, page 23 in [8]).](image)

As far as the porosity is concerned, Brace [6] observed that, for low porosity rocks (porosity less than 0.05), compaction is insignificant and dilatancy and elastic strain of minerals prevail. The onset of dilatancy occurs at about 50 to 75 per cent of the stress difference at fracture. In contrast, for high porosity rocks (porosity more than 0.05), both compaction and dilatancy can overlap and dilatancy begins closer to the peak stress compared to low porosity rock.
With the knowledge of the independent influences of confining pressure [8] and rock porosity [6] on compressive strength and volume change of rock, it would be interesting to explore the coupled influence of confining pressure and porosity on compressive strength and volume change of rock.

3 Pore-water pressure response during deformation

Understanding pore-water pressure response during deformation is of central importance for efforts to precisely characterize rock mechanical behaviour. The vast majority of triaxial tests have been performed under drained conditions where the internal pore-water pressure is maintained at a pre-determined value by controlling injection pressure [9]. Undrained deformation of rock in triaxial conditions allows the internal pore-water pressure to vary with increasing differential stress (i.e. during loading). Aldrich [9], from the results of triaxial experiments, observed that pore-water pressure increase is governed by the initial effective confining pressure and the maximum induced pore pressure increases with increasing effective confining pressure.

The maximum induced pore-water pressure will be dependent on the porosity of the material. However, a survey of the literature has revealed that this dependency is not yet completely understood. Therefore, this paper aims to provide some insights on induced pore-water pressure variation for relatively low and high porosity sandstone, tested under relatively high and low confining pressures.

4 Experimental work

Undrained triaxial tests were carried out on sandstone specimens having two different porosities (i.e. 13% and 18%) under 2 MPa and 25 MPa confining pressures. A pore-water pressure of 1 MPa was applied in all tests. Both types of sandstone are Argillaceous Quartz sandstone and sourced from New South Wales, Australia. The low porous sandstone was a medium-grained, grey-buff quartz sandstone with an average dry UCS of 54 MPa, whereas the high porous sandstone was a medium-grained, buff quartz sandstone with an average dry UCS of 57 MPa.

The sandstone blocks sourced from the quarry were cored at the Civil Engineering Laboratories, Monash University, to produce cylindrical cores 54 mm in diameter. The cores were then cut to limit the final core sample length to 108 mm, producing a length-to-diameter ratio of two. The two ends of
each sample were ground perpendicular to the long axis and parallel to each other, as required by the ASTM standards [10]. The prepared sandstone samples, with two different porosities for testing, are shown in Figure 2. In total eight tests were performed, comprising two replicates at each combination of sandstone type and confining pressure.

Conventional undrained, constant-strain-rate triaxial tests were performed on all the samples, during which the deviatoric stress was applied until the samples had failed. Stress-strain, pore-water pressure variation, and volume changes (as observed from flow of hydraulic oil from the pressure vessel) were recorded constantly during deformation, by means of an advanced data acquisition system. The arrangement of the triaxial set-up is shown in Figure 3.

5 Results and analysis

Axial stress-strain and pore-water pressure-axial strain relationships observed for both lower porosity and higher porosity sandstone specimens at low and high confining stresses are shown in Figure 4 and 5, respectively (only one curve is shown for each case, from the results obtained for two tested samples under each condition).

Figure 5 illustrates that in the higher porosity sandstone the induced pore-water pressure is higher than in the lower porosity sandstone for almost all the strain levels and the maximum pore-water pressure is gained more quickly in the higher porosity sandstone, under both 2 MPa and 25 MPa confining pressures. This suggests that the higher permeability of higher porosity sandstone allows an easier pressure communication within the specimen and also facilitates a higher pore-pressure induction compared to the lower porosity sandstone.
Mean stress (i.e. \(\frac{\sigma_1+\sigma_2+\sigma_3}{3}\)) vs. volumetric strain (as obtained from confining oil flow) variations were plotted for both the higher porosity and the lower porosity sandstone samples under 2 MPa and 25 MPa confining pressures (Figure 6).

**Figure 6**: Mean stress versus volumetric strain at (a) 2 MPa confining pressure and (b) 25 MPa confining pressure. The point 'C*' in figures refer to the onset of dilation

### 6 Conclusions and Future Work

A series of undrained triaxial tests were performed on sandstone specimens with two different porosities (i.e. 13% and 18%) under 2 MPa and 25 MPa confining pressures. A pore-water pressure of 1 MPa was applied for all the tests. The test results showed that the induced pore-water pressure was higher in the higher porosity sandstone compared to that in the lower porosity sandstone during most of
the test duration and that the maximum pore-water pressure was reached more quickly by the higher porosity sandstone. This was attributed to the fact that the higher permeability in higher porosity sandstone allows a quicker induced-pore-water pressure gain and higher pore-water-pressure induction compared to the lower porosity sandstone. The mean stress-volumetric strain relationships showed that the dilation is significant at lower confining pressures, but trivial at higher confining pressures. The onset of dilatancy did not vary considerably among the two types of sandstone. Perhaps if a larger confining pressure range and samples with considerably different porosity values were used, a noteworthy difference would have been observed.

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References

ROCK JOINT GEOMETRY: SIZE AND SURFACE

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New ideas for estimating a joint size distribution and determining the contact areas of a rock joint suggested by Rock Mechanics and Rock Engineering lab of Seoul National University are introduced. The former idea adopted a distribution-free estimation technique which does not require any assumption of the joint size distribution type or even of an upper limit on the joint size distribution [1]. The latter requires only three-dimensional surface coordinates at the initial stage before shearing, and a joint surface is modelled as a group of triangular planes of which contact condition is examined by calculating the relative displacement of both surfaces from their initial locations [2].

1 Introduction

3-D geometric modelling of rock joints requires information of joints such as the frequency, orientation, size, location, shape, and surface. The geometric information is essential to creating virtual joint network models for rock block stability analysis or for groundwater flow simulation, and to numerical joint surface modelling for virtual shear tests.

The field measurement of joint orientation, frequency, and size is biased due to constrained orientations and sizes of sampling zones. The process of removing the bias and guessing of the population values and distributions requires statistical estimation of the joint parameters [3]. The joint size is one of the most difficult geometric parameters to measure/estimate accurately [4,5]. This difficulty is caused to the facts that the estimation of joint size distribution is greatly affected by the joint shape and spatial distribution, and it is very hard to observe/measure the whole area of a rock joint.

Joint roughness is always a very important factor for understanding and estimating the shear behaviour of a rock joint. Although various methods such as joint roughness coefficient, statistical parameters, and fractal dimension [7] have been proposed to quantify joint roughness, it is still required to develop new ideas for better description of joint roughness. Considering that contact area of joint surface continuously changes during a shear process we need to define joint roughness and contact area dynamically by, for example, modelling a joint surface and monitoring the contact area at a certain level of normal and shear stress on the joint.

In this paper, new ideas for estimating a joint size distribution and determining the contact areas of a rock joint suggested by Rock Mechanics and Rock Engineering lab of Seoul National University are
introduced. The former idea adopted a distribution-free estimation technique \cite{1} while the latter modelled a joint surface with a group of triangular planes \cite{2}.

2 Distribution-free estimation of joint size distribution

2.1 Development of theory

The joint traces observed in a sampling window can be categorized into three types according to the number of end points located in the window \cite{6}: contained, dissecting, and transecting. Each type of joint trace has its own geometrical relationship between a joint trace length in a planar window and joint size in a 3-D rock mass space. The geometrical relationship is explained by the concept of the trace center zone and the joint center volume (Figure 1). A joint center volume of contained traces, $V_c$, for disc-shaped joints, for example, can be obtained by following equation:

$$V_c(s,l) = \sin \phi \left( \sqrt{s^2 - (l - dl)^2} - \sqrt{s^2 - l^2} \right) \left( W - l \cos \theta \right) \left( H - l \sin \theta \right)$$

(1)

where $\sin \phi \left( \sqrt{s^2 - (l - dl)^2} - \sqrt{s^2 - l^2} \right)$ is the thickness of the joint center volume and $(W - l \cos \theta)(H - l \sin \theta)$ is the area of the trace center zone.

![Figure 2](image)

Figure 2 (a) Trace center zone of contained traces of length $l$ and (b) joint center volume for a given trace length and joint diameter.

It is known that the joint shape will not be an issue in defining the joint center volume only if the relation between the thickness of the joint center volume and the trace length and/or joint size can be mathematically expressed. If the relation for a certain shape of joints is too complicated to be analytically obtained, the virtual generation of joints and their traces by a Monte Carlo simulation or Latin Hypercube can be adopted to define the joint center volume. In this case, the geometry of the sampling window, as well as the joint shape, cannot be an obstacle in defining the joint center volume because a joint center volume can be obtained by counting the joint centers in a virtual space for each joint size, whole trace length and partial trace length.

The number of contained traces whose length belongs to $l - \Delta l \sim l$ is $N_{l-\Delta l}^{c}$ and can be obtained by summing up the number of joint centers included in the joint center volumes corresponding
to the above trace length range and joint size equal to or longer than the trace length. The number of joint centers in a joint center volume is calculated by multiplying the joint center volume by the volumetric frequency of a corresponding joint size. The above explanation of the number of contained traces can be mathematically expressed as follows:

\[ N_i^{\varepsilon} = \lambda_s \sum_{i=1}^{S_i} V^c(s,l) c(s) \Delta s \]  

(2)

where \( V^c(s,l) \) is the joint center volume of contained traces whose trace length is \( l - \Delta l \sim l \) and joint size is \( s \). \( c(s) \) is the probability density function of the joint size. \( \lambda_s c(s) \Delta s \) is the volumetric frequency of joints whose size is \( s \), which indicates the number of joints with size \( s \) per unit volume of rock mass. \( S_i \) is the maximum joint size, but \( S_i \), \( \lambda_s \) and \( c(s) \) are not known in the site investigation stage and they therefore should be estimated by the estimation method described here. The same formulating process can be applied for the dissecting and transecting traces. For the sake of convenience in converting the formulas to computer code and to obtain a simplified expression, we change the variables in the subscript into ordinal numbers and reduce the number of coefficients as follows:

\[ N_i^{\varepsilon} \Rightarrow N_i^{\varepsilon} = \lambda_s \Delta s \sum_{j=1}^{n_s} \sum_{i=1}^{n_l} V^c(j,k,i) c(j) \]  

\[ N_i^{\varepsilon} \Rightarrow N_i^{\varepsilon} = \lambda_s \Delta s \sum_{j=1}^{n_s} \sum_{i=1}^{n_l} V^c(j,k,i) c(j) \]  

\[ N_i^{\varepsilon} \Rightarrow N_i^{\varepsilon} = \lambda_s \Delta s \sum_{j=1}^{n_s} \sum_{i=1}^{n_l} V^c(j,k,i) c(j) \]  

where the variable indices \( s \), \( l \) and \( l' \) are replaced with \( j \), \( k \) and \( i \), respectively. \( n_s \) and \( n_l \) are the number of divisions of the joint size and (whole) trace length obtained by dividing the maximum joint size and trace length by \( \Delta s \) and \( \Delta l \), respectively.

Denoting the numbers of three kinds of sampled joint traces as \( N_i^{c} \), \( N_i^{dn} \) and \( N_i^{t} \), the sum of the squared differences or errors between the sampled trace numbers and the theoretically calculated trace numbers can be expressed as follows:

\[ E^2 = \sum_{i=1}^{n_l} \left[ w_i^c \left( N_i^{c} - N_i^{c} \right)^2 + w_i^{dn} \left( N_i^{dn} - N_i^{dn} \right)^2 + w_i^{t} \left( N_i^{t} - N_i^{t} \right)^2 \right] \]  

(4)
where $n_l$ is the number of divisions of the partial trace length obtained by $L_i/L$, and $w_i^c$, $w_i^d$ and $w_i^t$ are the weights for the squared errors of the three trace types. Substituting equation (3) to equation (4) and partially differentiating it with respect to the joint size leads to the following equation:

$$[A_{pp}]c_j = [b_p]$$

where

$$[A_{pp}] = \sum_{i=1}^{n_l} w_i^c a_{ij}^c + \sum_{i=1}^{m} \sum_{k=1}^{n_l} w_i^d a_{ikj}^d + \sum_{i=1}^{m} \sum_{k=1}^{n_l} w_i^t a_{ikj}^t, \quad m_i = L(j, p, n_l)$$

and

$$[b_p] = \sum_{i=1}^{n_l} w_i^c N_i a_{ip}^c + \sum_{i=1}^{m} \sum_{j=1}^{n_l} w_i^d N_i a_{ip}^d + \sum_{i=1}^{m} \sum_{j=1}^{n_l} w_i^t N_i a_{ip}^t, \quad m_2 = L(p, n_l).$$

In equation (5), $m_1 = L(j, p, n_l)$ indicates that the least value among $j$, $p$ and $n_l$ is assigned to $m_1$, while $m_2 = L(p, n_l)$ indicates that the smaller value between $p$ and $n_l$ is assigned to $m_2$.

$[A_{pp}]$ is a $n_s \times n_s$ symmetric matrix and therefore, equation (5) can be solved by a matrix solver, such as a diagonally scaled conjugate gradient method, which was adopted in this study. The best solution is a solution that minimizes the sum of the squared error between the sampled numbers and the theoretically calculated numbers of traces. A recommended approach to determine the weights is to try various candidate sets of weights.

### 2.2 Verification

Verification with two steps was carried out to test the correctness of the suggested method. The first step is to compare the number of joint traces for each trace length range calculated by the suggested method with the number of the joint traces simulated by the Monte Carlo method using a predefined joint size distribution. This step confirms the validity of equation (3) and other formulae related to the joint center volumes of disk-shaped and rectangular joints.

The second step is to test the correctness of the estimation of the joint size distribution as well as the volumetric frequency. In this step, the number of joint traces at each trace length range is given by equation (3) for a predefined size distribution. This step removes any possibility of sampling error intervening in the joint size estimation, and therefore confirms the pure performance of the suggested estimation method.

Various predefined size distributions were adopted for disc-shaped and rectangular virtual joints: arbitrary (Figure 2(a)), uniform, ascending, descending, descending with double peaks (Fig. 2(b)), negative exponential and discrete bars.
All cases except for discrete bars showed a very good agreement between the theoretical distributions and estimated distributions with the largest error being 0.272%. The volumetric frequency, which is not a distribution but a single value, turned out to be estimated with even better precision than the size distribution. As for the case of joint diameter distribution consisting of discrete bars, reducing the weight increment from 0.2 to 0.01 was a very effective way of reducing the estimation error from 34% to less than 1.9%.

3 Surface modelling and determination of contact area

3.1. Representation of joint surface and its verification

The joint surface is commonly reconstructed from discrete data, although the topography is continuous in nature. In general, three-dimensional surface data are digitized on a XY grid by a laser or optical measuring equipment. Therefore, the elevation (Z) of each point can be expressed as a spatial function of x and y, Z(x, y). Figure 3 shows the opposite joint surfaces sheared by a horizontal displacement of dx, and their XY grids along which the elevation values are recorded. In the figure, Z\text{up}(x, y) and Z\text{low}(x, y) indicate the elevation functions of the upper and lower surfaces, respectively.

If we know the coordinates of both surfaces in the same system at a certain shearing stage, it is possible to determine whether a point on the surface is in contact or not by comparing the elevation values. For example, at point A or E in Figure 1b, the two surfaces may overlap and consequently be in contact, accompanied by elastic deformation and/or asperity failure, if Z\text{low}(x_A, y_A) ≥ Z\text{up}(x_E, y_E), where x_A = x_E and y_A = y_E. In practice, it is difficult to directly measure two joint surfaces at the same time during a shear test. However, the coordinates of the displaced surface can be estimated from the data measured at the initial stage and the relative displacements to the opposite surface, assuming that 1) the two joint surfaces are completely matched at the initial stage, 2) the asperities are not significantly damaged and 3) the elastic deformations of rock blocks and joint surfaces are negligible.

According to the first assumption, the elevation function of the upper surface at the initial stage Z\text{up, ini}(x, y) is nearly the same as that of the lower surface Z\text{low}(x, y).

\[ Z\text{up, ini}(x, y) \approx Z\text{low}(x, y) \] (6)
If the asperity failure and the elastic deformation can be ignored, the elevation of the displaced upper surface $Z_{up}(x, y)$ can be calculated from the relative displacements of the upper block to the fixed lower block, $dx$ and $dz$.

$$Z_{up}(x, y) = Z_{up, ini}(x-dx, y) + dz$$

(7)

Substituting equation 6 into 7, we obtain the following equation:

$$Z_{up}(x, y) = Z_{low}(x-dx, y) + dz$$

(8)

Then, the contact condition at a point $(x, y)$ can be defined as follows:

$$Z_{low}(x, y) \geq Z_{low}(x-dx, y) + dz: \text{Contact}$$

(9)

$$Z_{low}(x, y) < Z_{low}(x-dx, y) + dz: \text{Non-contact}$$

Figure 3. Schematic view of the surface grid information: (a) upper and lower joint surfaces sheared by $dx$ and (b) their XY grids along which the elevation values are recorded.

Examination of the contact condition based on the ‘point’ information can provide the locations of contact areas, but it remains difficult to describe their size and roughness. To overcome this deficiency, the joint surface was represented as a group of ‘plane’ elements, and each plane was examined in the contact condition. Because a plane is determined by three points that are not in a line, four neighboring points can define two triangular planes (e.g., $\Delta ABD$ and $\Delta BCD$ in Figure 3b). The contact condition for a plane is more complicated than that for a point because opposite planes can overlap if all three points do not satisfy the contact condition in equation 9. For example, when $Z(A) > Z(E)$, $Z(B) > Z(F)$ and $Z(D) < Z(H)$ or when $Z(A) > Z(E)$, $Z(B) < Z(F)$ and $Z(D) < Z(H)$, it is difficult to clearly determine the contact state between $\Delta ABD$ and $\Delta EFH$. To consider this problem, the coordinates of the center points of the planes were chosen as an alternative for the examination (equation 9). This
approach allows us to calculate the locations and sizes of the contact areas of a joint at any stage by back-analyzing the shear and normal displacements obtained from a laboratory shear test.

A direct shear test on a rock joint was simulated, and the contact area during the shear test was analyzed by using a bonded particle model, PFC2D. The locations of the contact areas monitored in the PFC simulations agreed well with those predicted using the proposed method, which suggests that the proposed algorithm is effective for determination of the contact areas at any of the shearing stages, in spite of the deformation and damage along a joint.

3.2. Application of the proposed technique

Direct shear tests were conducted on the tensile-fractured rock joints replicas under various loading conditions, and the proposed algorithm and surface roughness model were applied to the test results. The contact areas were analyzed in terms of their size, location and steepness, according to the shearing stages. To determine the effects of the normal load condition and the shear direction on the contact areas, two kinds of replicas with different joint shapes were prepared for the experiments (J1 with a rectangular joint and J2 with a circular joint). A granite block and a gneiss block were split to create a joint in the center of each. In the case of the gneiss, there was well-developed schistosity, and the joint was generated perpendicular to the schistosity, such that the anisotropy of the joint roughness could be maximized. The specimen J1 was tested under six different loading conditions, while the specimen J2 was sheared in four different directions.

Figure 4a shows the areal distribution of the micro-slope angles on specimen J1. The total elements covering the surface were sorted according to their micro-slope angles at 2° intervals, and the areas of the elements within each interval were summed. The micro-slope angles exhibited a bell-shaped areal distribution. Although the active part occupied slightly more area than the inactive part, the distribution was nearly symmetric about 0°. More than 99.9% of the micro-slope angles ranged from -60° to 60°, and the area-weighted mean and standard deviation were 0.85° and 14.54°, respectively. Figure 4b shows the distributions of the micro-slope angles on specimen J2 according to the shear direction. As the angle between the shear direction and the angle $\alpha$ increased, the areas with steeper apparent slopes increased, and those with low apparent slopes decreased, which suggests that this distribution may adequately qualify the joint roughness.
The variation of the contact areas with the shear displacement was examined based on the results of the CNL test on specimen J1 ($\sigma_n = 0.5$ MPa). The joint exhibited peak shear strength of 0.988 MPa when sheared by 0.615 mm, and it reached the residual shear state when it was sheared by 3.0 mm. Figure 5 shows the locations and micro-slope angles of the contact areas that were predicted at the following four shearing stages: the pre-peak ($dx = 0.2$ mm), peak ($dx = 0.615$ mm), post-peak ($dx = 0.8$ mm) and residual ($dx = 3.0$ mm) stages. In the figure, the white and colored regions denote the non-contact elements and contact elements, respectively. It is possible to observe the steepness of the contact areas which greatly affect the shear behavior of the joint.

Change in the contact areas was analyzed at shear displacements varied in 0.1 mm interval which were estimated from the results of the CNL (Constant Normal Load) test under normal stress of 0.5 MPa. The experimental results, such as the normal displacement and the coefficient of friction (the ratio of the shear stress to the normal stress, $\tau/\sigma_n$) are also presented. Here, the ‘contact area ratio’ is defined as the areal ratio of the contact elements to the total elements. The contact areas exhibited a gradual decrease in their shear displacement until the peak stage, which was mainly ascribed to the separation at the inactive zones. At the peak stage, the normal dilation was initiated, which led to a sharp drop in the contact area. Approximately 53% of the surface area remained in contact, supporting the normal and shear loads. The active zone was partially detached, and the inactive zone was partially in contact; this phenomenon was observed in elements with very low slope angles at the borders of both zones. After the peak stage, the contact area ratio decreased rapidly with increasing shear displacement, and few inactive elements came into contact until the residual stage. At the residual stage, only small fractions < 3 % were involved in contact.
The proposed method can be applied to prediction of the size distribution of aperture within a joint which has important implication in fluid flow and transport in rock mass. The aperture size can be calculated from the distance between upper and lower surfaces when the upper and lower surfaces satisfy the non-contact condition. Figure 6 shows the contour plots of the aperture size estimated at shear displacements of 0.2 mm and 0.8 mm. Most of aperture sizes were less than 0.1 mm at a small shear displacement, while the aperture sizes varied widely at shear displacement of 0.8 mm.

4 Conclusions

A method for estimating joint size distribution and volumetric frequency using 3 kinds of joint traces in a sampling window has been suggested. The concept of joint center volume has been proposed which are defined by shapes of joints and a sampling window, joint size, and trace length. This method can be applied to general cases of joint survey and joint geometry including relatively small sampling windows compared with the joints, and any shape of joints such as a circle, ellipse, rectangle or other polygon. The suggested method is a distribution-free estimation method since it does not require any assumption of the joint size distribution type or even of an upper limit on the joint size distribution.
A numerical method to determine the contact areas of a rock joint under shear and normal loads was proposed. In this method, a joint surface is reconstructed as a group of triangular planes and each plane is examined in the contact condition by back-analyzing the experimental results under the assumptions that two joint surfaces are completely matched at the initial stage and that the asperity damages is not significant. This method showed applicability to analyzing contact area of a joint under normal and shear load and to estimation of an aperture distribution within a rock joint.

References

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SENSITIVITY ANALYSIS OF ROCK MECHANICAL PARAMETERS AND THE 
APPLICATION IN ROCK ENGINEERING

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Abstract When the model for numerical analysis is specified, the validity of parameters will affect the results seriously. It is an effective way to decide the parameters in the model by back analysis using measured information. By checking the change of calculated displacement of a set of parameter obtained from the basic value with some degree of disturbance, this paper analyzed the effect of different rock mechanical parameters in permanent shiplock of TGP to the displacement of the measurement points. It is shown that the horizontal initial force, the elastic module, cohesion and friction of fresh and light weathered rock is of high sensitivity, it is also shown that the measurement points should be located at the position with large displacement in order to gain better analysis results for the parameter analysis.

Keyword rock mechanics; displacement back analysis; sensitivity analysis; optimal method

1 Introduction

As everyone knows, parameters of rock mechanics and initial in situ stress is the key to the success of the numerical method[1], attempts to resolve the difficulty of determining the design parameters related to rock engineering by improving testing technology and adopting new testing means has little effect. Back analysis based on the information obtained by in situ monitoring is a practical means developed to solve this question, and has made successful application in practice in rock engineering. In reference [2], a new back analysis method based on evolutionary neural network was proposed, it is an back analysis method which seeking a minimum objective function problem by evolutionary means based on the displacement information. For the optimized back analysis method, the sensitivity of the parameters to be back analyzed on the objective function must be analyzed thus determined the parameters which can be properly back analyzed. For sensitive parameters, it can be determined through the optimization back analysis method while the parameters with little sensitivity can not. In addition, the geological body studied by rock engineering is very complex, which cut by fault, joint, fissure, etc., and influenced by excavation and the external environment, which leads to the greatly increase of parameter number describing the rock engineering in numerical model. In this case, accurate determination of all the parameters is very costly and time-consuming, and it is not necessary, either by the back analysis or by indoor and field test, moreover, the more parameters to be back analyzed, the worse the results of uniqueness of parameters will be lead to [3~9].

In order to improve the efficiency and effect of back analysis and improve the precision of the result, it is necessary to analyze the sensitivity of the parameters to determine which parameters can be obtained by back analysis method, highlight the main factors, ignore the minor factors, thus make the parameter back analysis of complicated rock engineering problems possible [10,11].

2 Method of Sensitivity Analysis of Rock Mechanical Parameters

Sensitivity is the change of a variable caused by the change of the dependent variable. In back analysis
of rock mass engineering, the objective function often take the following form.

\[
\min F(X) = \sum_{i} (d_i - \hat{d}_i)^2
\]

Where the \( X \) are the series of parameter to be back analyzed, \( X = [E_1, \mu_1, c_1, \phi_1, E_2, \mu_2, c_2, \phi_2, ..., E_j, \mu_j, c_j, \phi_j, ...] \), where the \( E_j, \mu_j, c_j \) and \( \phi_j \) are the elastic modulus, Poisson's ratio, cohesion and angle of internal friction of the rock material \( j \) respectively, \( j = 1, 2, ... \), where \( j \) is the number of the type of rock material, \( \hat{d}_i \) is the measuring displacement of the in situ measuring point, \( d_i \) is the calculated displacement of the corresponding point, \( i = 1, 2, ... \), where \( i \) is the number of the measuring point.

By the formula (1), it can be seen that to determine the of rock mechanical parameters of \( X \) by back analysis, the objective function \( F(X) \) must have a certain sensitivity to every factor contained in \( X \), such as \( E_j, \mu_j, c_j, \phi_j \), etc. That is to say, when one element in \( X \) (\( E_j \) for example) changes, the calculated value of displacement in corresponding measuring point \( d_i \) may also have corresponding change, as well as the value of the objective function \( F(X) \), thus by continually modify \( X \) the value of \( F(X) \) will gradually approaching the minimum value, and eventually the optimal parameter sequence of \( X \) can be obtained. Therefore, if \( F(X) \) does not change or the change is small when a parameter from \( X \) changes, then this parameter can not be determined by back analysis using formula (1) as the target function.

So, sensitivity analysis can be carried out according to the following steps.

1. Selecting a series of parameter \( X_0 = [E_{10}, \mu_{10}, c_{10}, \phi_{10}, E_{20}, \mu_{20}, c_{20}, \phi_{20}, ..., E_{j0}, \mu_{j0}, c_{j0}, \phi_{j0}, ...] \) as the baseline parameters for the forward calculation, obtained the calculated value of displacement in each corresponding measuring point \( D_0 = [d_{10}, d_{20}, ..., d_{i0}, ...] \).

2. Change the \( r \)th parameter in the base series of parameters \( X \) (\( r = 1, 2, ..., \) where \( r \) is the number of parameters to be determined by back analysis simultaneously) by adding a disturbance, thus consists obtain a new series of parameters \( X_r = [E_{10}, \mu_{10}, c_{10}, \phi_{10}, E_{20}, \mu_{20}, c_{20}, \phi_{20}, ..., E_{j0} + \alpha E_{j0}, \mu_{j0}, c_{j0}, \phi_{j0}, ...] \) (where \( \alpha \) is the ratio of the disturbance to the corresponding parameter, usually \( |\alpha| < 1 \) ). Based on the new series of parameter \( X_r = [E_{10}, \mu_{10}, c_{10}, \phi_{10}, E_{20}, \mu_{20}, c_{20}, \phi_{20}, ..., E_{j0} + \alpha E_{j0}, \mu_{j0}, c_{j0}, \phi_{j0}, ...] \), forward calculation was taken, and calculated value of displacement \( D_r = [d_{1r}, d_{2r}, ..., d_{ir}, ...] \) corresponding to each measuring point was obtained.

\[
\Delta D_r = \frac{1}{n} \sum_{i=1}^{n} |d_i - d_i^0|
\]

3. Calculate \( \Delta D_r \), the average value of the difference between \( D_0 \) and \( D_r \), judge the degree of influence of the \( r \)th input parameter on the calculation results according to the size of \( \Delta D_r \). The bigger the \( \Delta D_r \) is, the higher the influence degree of the corresponding parameter on the calculated results (displacement).

3 Sensitivity Analysis of Rock Mechanical Parameters of section 17-17 relating to Three Gorges Project permanent shiplock

3.1 Engineering geology

The Three Gorges Project permanent ship was formed by excavation in the mountain, and the
excavation also lead to the formation of north shiplock slope in left line and the south shiplock slope in right line (the north slope and the south slope, in short). The section 17-17 is located at the head of the third shiplock room. The corresponding engineering geological zones are shown in Fig. 1. They consist mainly of a hard and intact rock mass slightly weathered in some places. It is suitable for deep excavation of high and steep slopes. The mechanical properties of the rock mass do not vary at the slightly weathered and fresh zones. The upper portion of the strata has completely and heavily weathered thin strata and then it is a moderately weathered zone. Therefore, mechanical parameters for these natural rock mass zones, such as the slightly and non-weathered zone, the completely and heavily weathered zone, and the moderately weathered zone are considered to be recognized.

The in situ stress is disturbed due to excavation of the slope, which generates unloading and relaxation zones. Because of excavation and blasting, a damage zone forms at the boundary of the excavation. Therefore, there are two other zones related to engineering activities, unloading and damaged zones, to be considered in the back analysis of displacement.

In this parameter sensitivity analysis, 20 parameters were considered, including the elastic modulus, cohesion and angle of internal friction of 6 kinds of rock mass (heavy weathered rock, moderate weathered zone rock, slightly weathered or fresh rock, damaged rock, unloading deformation rock and rock mass in F215 fault zone) and the constant ax, ay in equations (2) for calculating stress fields.

\[
\sigma_x = a_x + 0.01168H \\
\sigma_y = a_y + 0.03039H
\]

Where the unit of stress in equation (2) is MPa, H is depth below ground surface (m), and compressive stress is defined to be positive. X - lock transverse ( near South ), Y - upward. \( \gamma \) is rock mass density(106N/m3).

The measured displacements for sensitivity analysis were the cumulative displacement increments in the x-direction from excavation Step 7 to the end derive from the monitoring point TP/BM10GP01, TP/BM11GP01, TP/BM71GP01, TP/BM98GP02, TP/BM26GP02, TP/BM27GP02, TP/BM28GP02 and TP/BM29GP02, etc., as shown in Fig. 2.

Model and calculating conditions of the back analysis are as same as that in reference [11].
3.2 Plans designed for sensitivity analysis

According to the measured data in the Three Gorges area provided by relevant departments, the values of baseline parameters for the calculation were identified as shown in table 1.

Table 1 The disturbed parameters and their values related to each plan

<table>
<thead>
<tr>
<th>plan</th>
<th>Parameter</th>
<th>Original value</th>
<th>Value after disturbed</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Young’s modulus E of moderate weathered rock (GPa)</td>
<td>10.0</td>
<td>11.0</td>
</tr>
<tr>
<td>2</td>
<td>Young’s modulus E of damaged rock (GPa)</td>
<td>12.0</td>
<td>13.2</td>
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<tr>
<td>3</td>
<td>Young’s modulus E of unloading deformation rock (GPa)</td>
<td>20.0</td>
<td>22.0</td>
</tr>
<tr>
<td>4</td>
<td>Young’s modulus E of slightly weathered or fresh rock (GPa)</td>
<td>35.0</td>
<td>38.5</td>
</tr>
<tr>
<td>5</td>
<td>ax(MPa)</td>
<td>4.3982</td>
<td>4.83802</td>
</tr>
<tr>
<td>6</td>
<td>ay(MPa)</td>
<td>1.6628</td>
<td>1.82908</td>
</tr>
<tr>
<td>7</td>
<td>coefficient of friction of moderate weathered rock</td>
<td>1.30</td>
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<td>8</td>
<td>coefficient of friction of damaged rock</td>
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<td>1.54</td>
</tr>
<tr>
<td>9</td>
<td>coefficient of friction of unloading deformation rock</td>
<td>1.50</td>
<td>1.65</td>
</tr>
<tr>
<td>10</td>
<td>coefficient of friction of slightly weathered or fresh rock</td>
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<td>1.87</td>
</tr>
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<td>cohesion of moderate weathered rock (MPa)</td>
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<td>12</td>
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<td>1.1</td>
</tr>
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<tr>
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<td>Young’s modulus E of rock in F215 fault (GPa)</td>
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<td>coefficient of friction of heavy weathered rock</td>
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</tr>
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<td>cohesion of heavy weathered rock (MPa)</td>
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<tr>
<td>20</td>
<td>cohesion of rock in F215 fault(MPa)</td>
<td>0.5</td>
<td>0.55</td>
</tr>
</tbody>
</table>

Based on the series of the baseline parameters, one parameter was chosen for change by increasing the original numerical size of 10% as a disturbance (i.e., \(X_r = \{E_{10}, \mu_{10}, c_{10}, \phi_{10}, E_{20}, \mu_{20}, c_{20}, \phi_{20}, \ldots, E_{j0}+\alpha E_{j0}, \mu_{j0}, c_{j0}, \phi_{j0}, \ldots\}\), where in \(\alpha = 0.1\)), while the other parameters remained unchanged. The changed series of parameters (i.e., \(X_r = \{E_{10}, \mu_{10}, c_{10}, \phi_{10}, E_{20}, \mu_{20}, c_{20}, \phi_{20}, \ldots, E_{j0}+\alpha E_{j0}, \mu_{j0}, c_{j0}, \phi_{j0}, \ldots\}\), where in \(\alpha = 0.1\)) was regarded as a plan for sensitivity analysis. Every parameter in the
series was chosen to be changed for sensitivity analysis, thus 20 plans of sensitivity analysis were formed, as shown in Table 1.

3.3 Analysis of result

The relatively changes of displacement to original values in the monitoring points were shown in figure 3, and some following conclusions can be made:

Fig. 3 Relatively changes of displacement of the monitoring points

(1) The two components of the initial stress have significant effect on the calculating displacement, and the initial stress in the direction perpendicular to the axial direction (x-direction) is the most sensitive parameter of all, while the initial stress along the depth direction is of less sensitive. The reason is that the direction of displacement used to reflect the influence of the stress is consistent with the direction of the horizontal stress (the direction perpendicular to the axial direction, x-direction).

(2) The elastic modulus of slightly weathered or fresh rock has remarkable effect to the calculated displacement. It can be seen from the geology map of section 17-17 related to The Three Gorges Project that the chamber was formed by excavation in the slightly weathered or fresh rock mass, the excavation lead to the re-distribution of stress field and changed the properties of rock mass, formed the damaged zone and unloading deformation zone of rock mass, which affect the displacements of monitoring points TP/BM11GP01, TP/VM71GP01, TP/VM98GP02, TP/VM26GP02 and TP/VM27GP02, etc. greatly, for these monitoring points are located in the changed zone of rock mass.

(3) Although the rock mechanic parameters (i.e. elastic modulus, internal friction angle and cohesion) of damaged zone rock is less sensitive compared to that of the stress component in x-direction, it is also obvious. With the depth of excavation increases, the area of damaged rock mass and unloading deformation rock mass in the isolated rock mass and the two side walls of the shiplock development gradually to the lower section. When the excavation finished, the slope under the moderate weathered zone and the ship chamber near the free surface will be damaged and form damaged zone rock mass with certain thickness, which include the new fracture generated by blasting, opening and movement of the existing structure surface of rock mass, loose of rock mass, etc., and the strength of the rock mass is greatly reduced. Because some of the monitoring points (i.e. monitoring point TP/VM11GP01, TP/VM71GP01, TP/VM98GP02, TP/VM26GP02 and TP/VM27GP02, etc.) were located in the surface of the damaged zone of rock, the effect to the displacement in the
monitoring point caused by the damaged zone rock mass cannot be ignored despite of the smaller area the damaged zone rock mass is. In addition, after the completion of chamber excavation, almost all the rock mass in damaged zone changed into plastic zone, and the internal friction angle and cohesion changed too, these will also cause the change of displacement in the monitoring point.

(4) The rock mechanical parameters (i.e. elastic modulus, internal friction angle and cohesion) of moderate weathered zone rock mass and heavy weathered zone rock mass have little effect on the displacement on the monitoring point, that is to say they are less sensitive to the displacement on the point. It can be seen from the geology map that the heavy weathered zone rock mass and moderate weathered rock mass is located on the top of the rock mass with a smaller thickness, and excavation for chamber is in the slightly weathered or fresh zone rock mass which is remote to the moderate weathered zone rock mass and heavy weathered zone rock mass, so the changes of the rock mechanical parameters of moderate weathered zone rock mass and heavy weathered zone rock mass have little effect on the displacement on the monitoring point caused by excavation.

(5) The dip direction of F215 fault located in the south slope is basically as same as that of the south slope, and the distance between the fault and the free surface of south slope is short, so the strength of F215 fault will affect the calculation of displacement on the monitoring point located in the south slope, this is shown in figure 3. Figure 3 also shows that the displacement of monitoring point TP/BM98GP02, TP/BM26GP02 and TP/BM27GP02 were affected greatly by the elastic modulus of the F215 fault rock mass, while the displacement of other monitoring points were affected slightly. When the excavation finished, a little portion on the top of F215 fault rock mass which adjacent to the moderate weathered rock mass changed into plastic zone, and the other portion are still maintained in elastic conditions, so the internal friction angle and cohesion of F215 fault rock mass have very little effect on the displacement of the monitoring point.

(6) Monitoring points TP/BM70GP01 and TP/BM98GP02 were located in the vertical walls of the chamber, and its location is most close to the excavation area of chamber, compared to the other measurement points, so they are the most sensitive points to the change of parameters of rock mass. As can be seen from Figure 3, in all the changes of monitoring points caused by the change of horizontal ground stress with direction perpendicular to the shiplock axial, or the change of elastic modulus of slightly weathered or fresh rock mass, or change of rock mechanical parameters (elastic modulus, internal friction angle and cohesion) of damaged zone rock mass, the changes of measuring point TP/BM70GP01 and TP/BM98GP02 is the most obvious.

4 Conclusion and discussions

(1) Among the 20 mechanical parameters of rock mass to be recognized, the horizontal field stress component in direction vertical to the axial of the chamber, the elastic modulus of slightly weathered or fresh rock mass, the elastic modulus, cohesion, internal friction angle of damaged rock mass, the cohesion of rock mass in unloading deformation zone and the elastic modulus of rock mass in F215 fault have great sensitivity to the displacement on the monitoring point, as well as elastic modulus of rock mass in unloading deformation zone, the vertical component of field stress and the internal friction angle of rock mass in unloading deformation zone. So it is feasible to recognize these parameters by back analysis method based on the horizontal displacement of the monitoring points. At the same time, because changes of these parameters have greatly influence on the calculated displacement, the parameters recognized by back analysis method are reliable.
(2) Parameters (such as the elastic modulus, cohesion, internal friction angle of heavy weathering rock mass, the cohesion of slightly weathered or fresh rock mass, etc.) of low sensitivity cannot be recognized by the back analysis method using formula (1) as the objective function, they should be recognized by other methods.

(3) Location of monitoring points affect the result of the calculated displacement greatly, therefore, in order to increase the sensitivity of the parameters, the monitoring points should be located in the area where the excavation may cause notable change of displacement.

Reference
MECHANICAL CHARACTERISTICS OF GRANITE UNDER HIGH TEMPERATURE

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The laboratory tests are made for studying on mechanical characteristics of Linyi granite, Shandong at room temperature to 800°C. The variations of peak stress, peak strain, elastic modulus and stress-strain curves are analyzed at high temperature as well as after high temperature. Studying results have shown that peak strength and elastic modulus decrease continuously with increasing temperature, while peak strain increases overall at high temperature. The mechanical index of granite after high temperature decreased significantly with increasing temperature, but generally better than that at high-temperature. The failure type of Linyi granite shows distinct brittle failure when temperature is below 800°C.

1 Introduction

As high-quality building materials, granite has high hardness and wearability, which especially has been widely used as an ideal place for underground storage of nuclear waste and radioactive materials as well as salt rock recently. Therefore, the characteristics of strength and deformation of granite under high temperature are directly related to the leakage of radioactive waste and contamination.

In recent years, many fruitful results have been carried out about physical and mechanical properties of rock under high temperatures, among which laboratory tests are the most common methods. Chen et al. [1] analyzed the influence of different high temperatures on peak stress, peak strain, elastic modulus and stress--strain curves of granite. Ferrero et al. [2] studied two Italian marble (black, white marble) in the temperature range from 230°C to 600°C, measured the density of the two marble at different temperatures and investigated the impact of density on the mechanics behavior of rock. Luo et al. [3] studied the mechanical properties of mudstone in the underground coal gasification process. Liang et al. [4] found that plastic strain increases gradually and strain-softening behavior of the specimens becomes increasingly evident through a series of physical and mechanical tests on salt rock at different temperatures. Wan et al. [5] researched the thermal deformation and failure characteristics of large size granite at high temperatures and triaxial stresses by utilizing the 20 MN servo-controlled triaxial rock testing machine. Wu et al. [6] studied the apparent shape, peak stress, peak strain, modulus of elasticity, Poisson’s ratio and stress-strain curves of Jiaozuo sandstone after undergoing different high temperatures. Xu et al. [7] investigated the relationships between mechanical characteristics of rock and microcosmic mechanism at high temperatures as well as the stress-strain behavior of granite under the action of temperatures ranging from room temperature to 1200°C. Zhang et al. [8] analyzed the variations of uniaxial compressive strength, elastic modulus, longitudinal wave velocity, cut-slip strain and the effects of thermo-mechanical coupling under uniaxial compression at high temperature. Zhang et al. [9] studied the mechanical properties of marble, limestone, and sandstone as well as the stress-strain curves, the varying characteristics of the peak strength, the peak strain and elastic modulus under the action of temperatures ranging from room temperature to 800°C. Zhu et al. [10] researched the variation of peak stress, peak strain and elastic modulus and the relationship between longitudinal
wave velocity and peak stress and peak strain of tuff, granite and breccia after different high temperatures based on the uniaxial compression test. However, there are few studies about the comparison of physical and mechanical properties at high temperature and after high temperature.

In this paper, the laboratory tests are made for studying on the variations of peak stress, peak strain, elastic modulus and stress-strain curves of Linyi granite at room temperature to 800°C and after high temperature.

2 Experimental description .

2.1 Specimens

Granite specimens used in our investigation were taken from Linyi, Shandong, and their mineral components are feldspar, amphibole, quartz and small amounts of other minerals. The average density of granite is 2.92g/cm³. Rock specimens were machined into cylindrical specimens which were about 45 mm in length and 20 mm in diameter. The specimens were dense and there were no macro-cracks and pores. Fig.1 is the photo of part granite specimens before heating.

![Fig. 1 Photo of part granite specimens before heating](image)

There are 37 granite specimens used for the experiment. These specimens were divided into eight groups, of which the temperature ranges are at 20°C, 200°C, 400°C, 600°C, 800°C and after 400°C, 600°C, 800°C. The information of specimens used for the experiment is listed in Table 1.

<table>
<thead>
<tr>
<th>Temp.</th>
<th>Number</th>
<th>Quantity</th>
</tr>
</thead>
<tbody>
<tr>
<td>20°C</td>
<td>A-1-A-5</td>
<td>5</td>
</tr>
<tr>
<td>200°C</td>
<td>B-1-B-5</td>
<td>5</td>
</tr>
<tr>
<td>400°C</td>
<td>C-1-C-8</td>
<td>8</td>
</tr>
<tr>
<td>600°C</td>
<td>D-1-D-5</td>
<td>5</td>
</tr>
<tr>
<td>800°C</td>
<td>E-1-E-4</td>
<td>4</td>
</tr>
<tr>
<td>400°C  H</td>
<td>F-1-F-4</td>
<td>4</td>
</tr>
<tr>
<td>600°C  H</td>
<td>G-1-G-3</td>
<td>3</td>
</tr>
<tr>
<td>800°C  H</td>
<td>H-1-H-3</td>
<td>3</td>
</tr>
</tbody>
</table>

(Note: 400°C H, 600°C H and 800°C H denotes respectively after undergoing 400°C, 600°C and 800°C high temperature.)

2.2 Equipments

The tests were mainly conducted by MTS810, MTS815 hydraulic servo system and MTS653.04 high temperature furnace from room temperature(20°C) to 800°C. The temperature of the furnace can be as
high as 1400±1℃ and its heating rate is 100℃/min. The time to reach highest temperature is less than 15min.

2.3 Research process

a) First of all, rock specimens are numbered and divided into different groups according to the temperature ranges. The number of rock specimens in each group is usually 3 to 5. If a set of experimental data discrete, the number of test rock specimens should be increased.

b) When specimen is heated to the target temperature, keep this temperature constant for 15min, and then start uniaxial compressive test for specimen. Or conduct uniaxial compressive test until specimen cooling to room temperature.

c) Record and sort out the related parameters of granite which obtained from the experiment.

The displacement control mode is used in uniaxial compression test of granite and the deformation rate is 0.003mm/s.

3 Results and analysis

Through the analysis of the uniaxial compression test data, we can obtain mechanical parameters of Linyi granite such as stress-strain relationships, peak stress, peak strain and elastic modulus at different temperatures and then compare these mechanical parameters with the changing temperature.

3.1. Stress-strain curves

The representative stress-strain curves of granite at different temperatures are shown in Fig.2.

![Fig. 2 Representative stress-strain curves of granite at different temperatures](image)

We can summarize the following characteristics from Fig.2. The stress-strain curves of the granite have a similar distribution in each temperature stage, which approximately goes through four stages containing compaction stage, linear elastic stage, plastic deformation stage and destruction stage. When temperature is at 20℃~800℃, plastic deformation stage of granite in the stress-strain curves is not obvious and the failure type of granite shows brittle failure.

In contrast to the stress-strain curves at different temperatures, we can draw that peak stress of the granite shows a significant downward trend, indicating that the deformation increases and the plasticity enhances with the temperature increases from 20℃ to 800℃. When stress reaches the peak value, rock specimens quickly rupture and the final break point was a brittle failure. The slope of the curve in the
linear elastic stage decreases with increasing temperature, indicating that elastic modulus decreases with temperature increases. The failure mode does not present plastic flow characteristics below 800°C, so we can speculate that the threshold temperature which brittle failure transforms to ductile failure is above 800°C.

The representative stress-strain curves of granite after undergoing different high temperatures are shown in Fig.3.

It can be seen from Fig.3 that the brittleness of granite weakens, the ductility enhanced, peak stress decreases and the axial strain increases with the temperature increases after high temperature. However, the granite does not present apparent plastic and post-peak behavior and the failure type of granite shows also brittle failure up to 800°C. This indicates that if there appears apparent plastic deformation stage for Linyi granite, the temperature should be above 800°C.

Comparing the stress-strain curves of granite at high temperature and after high temperature, we can obtain that both go through four stages in which the plastic deformation is not obvious. The reason is that granite has high hardness and is easily weathered which indicates that stress decreases rapidly after reaching the peak stress while strain changes little.

3.2. Peak stress

Variation of peak stress for Linyi granite at different temperatures and after high temperatures and their comparison are shown in Fig.4.
Variation of peak stress of granite with temperature changes from 20℃ to 800℃ is shown in Fig. 4 (a). It can be seen that peak stress presents a large discrete which is due to the structural differences (heterogeneity), the size effect, the end effect and the precision of processing factors of rock specimens, but we can still obtain a strong regularity by the observation of change of average peak stress. Peak stress decreases gradually from 20℃ to 800℃ in real-time high temperature and minimizes at 800℃, which decreases from 114.27MPa to 53.61MPa, a reduction of about 53.08%. Although peak stress presents a general downward trend, the change of uniaxial compressive strength is not very obvious below 400℃. When the temperature exceeds 400℃, there is a sharp decline in peak stress of granite with temperature increases and the mechanical properties shows a rapid deterioration.

Variation of peak stress of granite after high temperature is shown in Fig. 4 (b). The average peak stress from 400℃ to 800℃ shows a linear decline trend, a reduction of about 34.28 % and 38.38% respectively.

Fig. 4 (c) shows peak stress of granite after 400℃ and 600℃ is higher respectively than that at 400℃ and 600℃, which may be due to the reason that part of internal cracks healed resulting the intensity increment under the influence of the surrounding environment when granite specimens cooled. The average peak stress of granite at 800℃ is high than that after 800℃. The reason may be that the internal structure of granite at 800℃ have produced more damages, the intensity have been unable to recover in spite of cooling.
3.3. Peak strain

Variation of peak strain for Linyi granite at different temperatures and after high temperatures and their comparison are shown in Fig. 5.

It can be seen from Fig. 5 (a) that peak strain of granite shows a certain discrete while the overall trend is that peak strain increases with temperature changes. The average peak strain of granite
increases from room temperature to 400°C, of which increases slightly from 200°C to 400°C. The average peak strain of granite decreases slightly from 400°C to 600°C, then it begins to increase from 600°C to 800°C and finally reaches the maximum at 800°C. Overall, peak strain of granite increases at high temperature, of which there are a little fluctuation between 200°C to 400°C which may be due to the pore structural difference of rock specimens.

It can be seen from Fig.5 (b) the variation of peak strain for granite after high temperature. The average peak strain of granite decreases from 400°C to 600°C while it increases from 600°C to 800°C and it reaches the highest, namely 1.04 ×10^-2.

Fig.5 (c) shows that the average peak strain of granite at 400°C, 600°C and 800°C is higher respectively than that after 400°C, 600°C and 800°C.

3.4. Elastic modulus

Variation of elastic modulus for Linyi granite at different temperatures and after high temperatures and their comparison are shown in Fig.6.

It can be seen from Fig.6 (a) that there is a strong regularity that elastic modulus decreases with increasing temperature. The average elastic modulus of granite reduced from the highest point to the lowest from room temperature to 800°C, namely from 19.18GPa at room temperature to 6.64GPa at 800°C, a reduction of about 65.38%. We can observe that the mechanical properties of granite shows a trend of continuous deterioration in the real-time high temperature.

From Fig.6 (b) we can observe variation of elastic modulus for granite after high temperature. The average elastic modulus shows a downward trend from 400°C to 800°C, among which elastic modulus shows a moderate decline from 400°C to 600°C and it decreases rapidly from 600°C to 800°C and it reaches the minimum at 800°C. Elastic modulus after 400°C, 600°C and 800°C is 12.27GPa, 11.75GPa and 5.76GPa respectively, a decline of about 4.24% and 50.98%.

Fig.6 (c) shows the average elastic modulus of granite after 400°C and 600°C is higher respectively than that at 400°C and 600°C. The average elastic modulus of granite at 800°C is high than that after 800°C, among which the maximum difference appears at 600°C.
4 Conclusions

(1) The stress-strain curves of granite at high temperature and after high temperature are broadly similar which go through four stages and the plastic stage is not obvious. The failure type of granite shows brittle failure.

(2) Peak stress and elastic modulus of granite show a downward trend of continuous deterioration both at high temperature and after high temperature. In addition to the peak stress and elastic modulus at 800°C are lower than that after 800°C, the mechanical properties of granite after high temperature are superior to the real-time high temperature.

(3) Peak strain of granite increases overall with increasing temperature in spite of a little fluctuation from 400°C to 600°C. The average peak strain of granite at high temperature is higher than that after high temperature.

References

LEAN TUNNELING FOR THE 21ST CENTURY: A NEW CONCEPTION AND METHODOLOGY

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This paper presents a new conception—lean tunneling, which is contrived to meet increasingly tough challenges in the 21st century. Lean tunneling refers to integrating tunneling principles into shield tunnel construction, in order to consider the characteristics of shield tunneling process, five basic principles of lean thinking are elaborated and seven types of wastes are pointed out. Then a case study is given to demonstrate the initiative application of lean tunneling in a river-crossing tunnel.

1 Introduction

Lean thinking originates from the Toyota Production System (TPS). In 1990, James P. Womack and Daniel T. Jones published the first book to reveal TPS\textsuperscript{[1]}. This book tells the story of mass versus lean production and shows why lean is superior. It tells not only why Toyota wins but how any organization embracing the complete system of lean production can also win. Right after publication, the book caused a stir and positive reaction in the western automotive industry. In 1996, the authors of this book published another classic book ‘Lean Thinking: banish waste & create wealth in your corporation’ \textsuperscript{[2]}. The concept of lean thinking was firstly put forward. Five inherent principles in any lean system and seven types of waste were summarized. Lean Thinking clearly demonstrates that those simple ideas can breathe new life into any company in any industry in any country. With in-depth studies of more than fifty lean companies in a wide range of industries across the world, the book also provides a step-by-step action plan. Since then, the philosophy of lean thinking has spread to various areas and sectors.

Meanwhile, during the past two decades, studies have been carried out to explore the possible applicability of lean thinking in the construction industry. Traditionally, construction industry is characterized by intensive labor work, high rates of rework, and high risk of not meeting customer needs. Researchers have been hoping that lean thinking could bring this industry into a new era.
To make lean thinking more adaptable to construction, some supportive theories have been set up such as Transformation-Flow-Value theory [3, 5, 6, 7], Last Planner System [4]. Lean Construction has been successfully applied in different civil projects like commercial, residential buildings and bridges. However, we can hardly find research on the application of lean thinking in the underground construction. In this paper, the concept of Lean Tunneling is created by us and the adaption of lean thinking in tunneling is elaborated, and a case study is given as well.

2 Lean Tunneling

In this section, the limitations of current research work are briefly explored and contrast them with commercially accepted development practices. We then discuss anecdotal evidence that supports an iterative development process incorporating client-developer joint exploration of partial designs to facilitate the development of client understanding of their needs. Lean tunneling could be defined as ‘a philosophy that integrates lean thinking with tunnel construction, tries to eliminate all the waste during construction, sets the smooth and quick excavation of shield tunnel as the mission, optimizes the whole processes, changes the traditional push style construction into a pull & push style method driven by shield excavation, provides construction with the appropriate amount of supply at the right time, right place, realizes continuous improvement through feedback mechanism’.

Five principles of lean tunneling can be elaborated as follows.

2.1 Value

Specifying value correctly is primary task of lean thinking. In lean thinking, value is defined from the standpoint of the end customer. Only the product of an upstream satisfies the exact need of the last step, can it be considered as value adding.

Traditionally, accomplishment of every step in construction is value adding. Local optimization is encouraged while the equally vital goal of work flow management is neglected. Every workman tries to finish as much work as possible without considering the real need of downstream steps. When the demand of the downstream changes, those finished work may need to be reworked, which is not only a huge waste of time and labor, but also results in the delay of the whole project.

In lean tunneling, shield excavation is considered as the key customer of all steps. Though shield excavation is not the end step of tunnel construction, it acts as the constraint and of most importance. If the shield machine stops, all the other works cannot perform properly. So in lean tunneling, we define value as ‘procedures carrying out at a right time, right place, and meeting the precise demand of shield excavation’.

2.2 Value Stream

After specifying the right value, the next step is to identify the value stream for the tunneling and remove wasted steps that don’t create value but do create wastes. In lean tunneling, value stream is all the specific actions required to provide the materials that shield excavation needs. Thus all the steps that serve the advance of shield tunneling are components of value stream. Steps that do not add value, that represent waste or that shield tunneling does not want and would not pay for are not part of the value stream. In lean tunneling, we can evaluate each step and figure out the wastes and the steps can be categorized three types as shown in Figure 1.
Figure 1 Schematic of different types of work

- **Value-adding work**: those that add value, such as shield advance, rebar planting, road deck construction etc.

- **Assistant work**: those that do not add value but are requisite, such as back-fill grouting, equipment maintenance, quality test of component etc.

- **Waste**: those that do not add value and can be eliminated, such as inventories, delay (waiting time), unnecessary processing, overproduction, chaotic transportation, motions and defective products.

As shown in Figure 1, with respect to a typical project, wastes often take up a large part of all activities. The goal of lean tunneling is to keep eliminating wastes and achieve “perfection”.

### 2.3 Flow

All activities, including ordering, supply chain, production and service, can be transformed into a flow. In an ideal lean tunneling system, all the necessary steps required to complete the task are arranged in a stable and continuous flow. There are no waste steps, no disturbance and no queuing in it.

The objective of flow should be specified based on the value and value stream of lean tunneling. Flow in lean tunneling is to make the value stream of tunnel construction flow smoothly and rapidly on the demand of shield excavation. Those indispensable steps are linked up in an optimized way to form a non-disruptive and non-waiting flow so that waste and barriers can be easily noticed and then removed. The traditional construction’s pursuits of maximizing the profits of a single step and different steps standing against each other must be abandoned. All the relevant processes of value stream along the construction line should be integrated. The optimized flow scenario can only be reached with the participation of all steps.

The best flow condition cannot be created just at the beginning of a project. The flow needs to be improved repeatedly after cycle as we will discuss in section 2.5.

### 2.4 Pull

In traditional ‘push’ system, large numbers of raw materials are transported to the site storages based on the production planning or forecasts. Then various processes begin to manufacture according to their own schedule. Their products again become inventories that wait to be consumed on an uncertain time. On the basis of forecasts, workmen ‘push’ the product or material along the production line to the next operation. ‘Push’ often ignores the changing demand of the downstream and tries to solve the problem with mass storage. When the downstream’s requirement changes, which is often the case in tunnel project, the standard tunnelling process cannot satisfy the demand, then rework and
wastes occur. Besides, ‘push’ lacks cooperation between different departments. Errors tend to accumulate along the line. The system can’t give timely response to emergency situations.

On contrast, lean ‘pulls’ value to flow from rapid-response value streams when needed. Pull is an important conception in lean tunneling. The system produces only what the end tunneling demands. The upstream step produces material or provides service only by receiving demand signal from the adjacent downstream steps. That means all the construction processes are in coordination with actual demand and consumption. Comparing with “push” system, a “pull” one is more flexible and has better adaptability. Pull system can minimize wastes. Since each upstream’s process or services are provided according to the real-time needs of the downstream, the applicability can be ensured and errors can be found and rectified in time. Wastes caused by inventories are reduced in the meantime.

As afore-mentioned that shield excavation is not the end progress, but it is of the utmost importance and acts as the constraint. The excavation is often affected by unexpected situations, such as the sudden change of geological conditions or shield machine failure. A smooth flow depends largely on the advance of shield machine. Taking this into consideration, we can define a “Pull & Push” system in lean tunneling. The level production schedule is sent to the shield excavation process. The shield excavation pulls the implementation of upstream processes like the preparation of rebar, linings and slurry. Upon receiving information feedback, upstream steps start to provide the shield excavation with right amount of product and service at the right place meeting its real-time requirement. Meanwhile, shield excavation pushes the execution of downstream such as concrete casting and electromechanical installation so that those works are in pace with shield advance.

2.5. Perfection

The ideal state of lean tunneling cannot be reached right after implementing lean tunneling measures. While pulling and pushing the whole processes with shield excavation, the waste and barriers that cumber the smooth and quick flow will be exposed. By eliminating new discovered wastes, continuous improvement can be achieved. In the pursuits of perfection, we should reduce waste, and optimize the entire flow cycle after cycle.

Perfection is a state that the whole stream delivers pure value continually with zero waste. It is the mindset of lean tunneling. In tunneling, the accomplishment of each construction section is defined as a cycle. Similar construction processes are repeated again and again in different sections with the advance of shield machine, so it creates a great condition to improve the flow after each cycle. The wastes exposed in a cycle should be addressed in the next one, and every new cycle should flow more smoothly than previous ones.

PDCA (plan-do –check –action cycle) or Deming circle is applied to realize continuous improvement and perfection. Proposed by W. Edward Deming in the 1950’s, PDCA is widely used in business to identify sources of variations and form a continuous feedback loop. The four-step method can be clearly demonstrated as shown in Figure 2.
3 Case Study

3.1. Project Background

The project studied here is Qianjiang river-crossing road tunnel in Hangzhou, with a length of 3.75km and diameter of 15m. A Φ15.43m large slurry shield tunnel boring machine (TBM) is applied to construct two tunnels in sequence. The first tunnel is excavated from south to north and then second one from north to south. Due to the limitations of site space, supply chain, etc., construction facilities have to be located at the south bank of the river. So when excavating the second tunnel, the first one will be used to transport different components. To meet the requirement of time limitation and explore better construction method, lean tunneling is introduced and initatively applied in certain aspects of this project.

3.2. Site layout

Good site planning is a prerequisite for creating smooth flow and avoiding chaotic transportation. Figure 3 is the schematic of the construction site layout. Considering the rectangular shape of the site, different workshops and inventories are arranged along the main construction passage. The layout eliminates unnecessary road turns so it is convenient for big truck. Furthermore, the truck can get all the material along the main passage in a short route.
3.3. Synchronous construction technology

Combined with previous engineering experience, a synchronous construction technology of internal construction is employed. The prefabricated road element erection and TBM advancing are synchronous. As shown in Fig.4, internal construction is divided into five construction sections. Each section is 30m long. Right upon TBM starts advancing, it pulls the work in sections behind the excavation face. In parallel with bored tunnel construction, section A1 performs chiseling and erection of prefabricated road element. Section A2 does the work of rebar planting. A3, A4 carry out concrete casting of ballast block and corbel respectively. And in Section A5, the road deck structure is constructed simultaneously. Along with shield advancing, all the processes in a section will be done in sequence. Practice in this project has proven that the synchronous construction is efficient, and creates a clear and smooth flow in tunnel. Any delay in any section can be noticed by naked eyes and it is easy for the crew to see the progress of their colleagues so they will know if they are in pace with the whole procedures.

Besides, the erected prefabricated road element provides a special transit passage for segments, prefabricated road elements and materials. Thus, the components can be transported directly to the working face by certain transporting vehicles.

![Figure 4 Schematic of internal construction](image)

3.4. Buffer Inventories

Here the lining segments for tunnel ring are taken as an example. Segments are produced at a firm 15 minutes drive away from the construction site. Appropriate number of lining storage at site is necessary to act as buffer. Based on the average daily shield advancing speed, the site has a storage capacity of 60 segments (6 rings) acting as a buffer, as shown in Fig.6. Every day, after receiving the daily plan, the vendor will deliver linings to replenish the reserve. The practice has proved that 60 linings buffer are enough to prevent possible delay, and more inventories may not be beneficial and be a waste of space. The production and storage of other components, such as prefabricated road element and rebar, are also planed likewise and synchronized with the pace of tunnel excavation.

4 Conclusions

A new conception and methodology, lean tunnelling, has been firstly presented in this paper. Lean tunnelling integrates lean thinking with tunnel construction and shows good adaptability. In the case study, lean thinking is adopted to optimize site layout, synchronous construction technology and inventories for the Qianjiang river-crossing tunnel. During the engineering practice, it has been proven that those optimizations can create a better flow of the project and increase the efficiency of construction. However, in this project, lean tunneling is only applied to improve certain plans and steps,
not fully involved in the overall management of the project. Further in-depth studies and more practical engineering cases are therefore required to complement and improve the lean tunneling theory.

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CASE STUDIES ON EQUIVALENT ISSUE OF SOIL SLOPE STABILITY ANALYSIS METHODS

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Generally, slope stability evaluation involves two coupled tasks: locating the critical slip surface (CSS) and calculating its corresponding factor of safety (FOS). In this article, the slope stability analysis methods are divided into three categories: limit equilibrium methods (LEM), finite element methods (FEM), as well as combined methods of FEM and LEM. Additionally, different types of these approaches have a significant impact on the location of CSS and its corresponding FOS. Following the review of equivalent issues between FEM shear strength reduction method (FEM-SSR) and LEM, a comparative research on the location of CSS and its FOS was performed based on the three types. The results showed that: First, the equivalence between FEM-SSR and LEM does not strictly exist in theory. No matter the method is, such as FEM, LEM or combined methods, is considered, they always have their own premises and limitations of application. These factors hardly determine and achieve the equivalent. Thus, they can not be completely replace each other. Secondly, the FEM-SSR has inherent defects, which are featured by a notable position drift of CSS occurred after strength reduction. Although there is no theoretical guarantee for the CSS obtained by FEM-SSR, the advantages of its convenience and concision can remedy its disadvantages of lacking to some extent theoretical rigor. Therefore, the analysis result of FEM-SSR is still an important reference for engineering practices. Finally, base on the combined methods of FEM and LEM, a certain kind of advanced non-linear optimization methods such as swarm intelligence methods can be considered as a promising direction for further research of slope stability evaluation problem.

1 Introduction

Soil slope stability evaluation is an old and active problem in geotechnical engineering research fields. It has been widely accepted that the most fundamental concept in slope stability evaluation is the factor of safety (FOS). Generally, the computation of the FOS and the location of the critical slip surface (CSS) are coupled tasks in slope stability evaluation. Among all the possible failure surfaces, each of them has its own FOS and there must be a minimum FOS existing. So, the minimum value and its corresponding surface are respectively defined as FOS and CSS of the slope. In this sense, slope stability analysis is a process of identifying the most dangerous potential failure surface.
From the perspective of mathematics, the effort to assessing stability of the slope is a non-linear optimization problem. That is, under certain constraints, searching for the minimum value of a particular non-linear performance function. Obviously, the performance function can be defined as FOS or its derivatives.

Although, there are various types of approaches for slope stability assessment have been proposed in the past decades, in general, they can be divided into three categories: the limit equilibrium method (LEM), finite element methods (FEM) (FEM, refer to the general numerical methods, including the finite difference method), and the combined methods of FEM and LEM. Briefly, the third method is based on the stress field obtained by FEM, and it defines the FOS as the ratio of shear strength and actual shear stress along a slip surface [1].

Different types of these approaches have a significant impact on the location of CSS and its corresponding FOS. The following review is of equivalent issues between FEM shear strength reduction method (FEM-SSR) and LEM. A comparative research on the location of CSS and its FOS was performed based on the three methods. The present paper is geared towards revealing a fact that the equivalent between LEM and FEM practically does not exist.

2 Debate on the Equivalence of Slope Stability Analysis Methods

It seems that the FEM-SSR have the ability to locate the CSS automatically, while other evaluation methods (i.e. LEM and combined methods) are facing the problem of searching CSS. Comparing the combined methods with the LEM, the former have more theoretical advantages. By using the stress field obtained from the FEM, the combined methods can overcome the unreasonable assumptions of LEM. For each slip surface in the stress field, the accuracy of FOS is ensured by the FEM, while the global optimization performance (accuracy and efficiency) for searching the CSS is subject to the capability of the selected nonlinear optimization method. Additionally, the swarm intelligence methods are currently ascending in the non-linear optimization field, and have obvious advantages approaching various optimization problems.

No matter the LEM or combined methods of LEM and FEM is concerned, the searching algorithms is seemed to be difficult and complicated. Precisely, the considerable difficulty of the FEM-SSR makes it booming in the recent years [2-8].

Although the FEM-SSR has a wide adaptation to automatically obtain the CSS, it is consistent with the result deduced from the other two types of methods, is still doubtful. The subject of this issue is the equivalence problem between FEM-SSR and LEM.

This controversial issue has important theoretical significance and practical value. If the equivalence is present, then we do not have to adopt a more complex and cumbersome searching method, but should boldly use the FEM-SSR to replace the former. Moreover, the focus on searching the CSS leads to lose its meaning. However, such is not the case. The field regarding how to efficiently find out the CSS is one of the forefronts of slope stability research.

Although many scholars believe that the equivalence exists, Zheng et al. (2006) [9] make a classification and summary for different definitions of FOS. Furthermore, he pointed out the widespread former proof of the equivalence is not thorough and has some defects. From the perspective of information theory, Liu (2010) [10] proved that searching the CSS belong to NP-Complete problems,
which is one of the key concepts in computational complexity theory, moreover, FEM-SSR is not equivalent to LEM due to the inherent properties of NP-Complete problems.

Currently, the debate continues. It is necessary to perform a case study to compare some characters, such as the location and shape of CSS as well as the corresponding FOS, among three types of slope stability assessment approaches. From the results, the controversial issue can be clear.

3 Case Studies

3.1 Case Profile

The test example is taken from the EX1(C) of ACADS slope stability programs [11]. The mechanical properties of the soil are shown in Table 1. The stability evaluation results are shown in Table 2 and Fig. 1.

Table.1. Mechanical properties of the soil of ACADS EX1C problem (after Donald and Giam 1992)

<table>
<thead>
<tr>
<th>Soil Layer</th>
<th>Cohesion C(kN/m²)</th>
<th>Friction φ(°)</th>
<th>Weight γ(kN/m³)</th>
<th>Elas. Modu. E(kN/m²)</th>
<th>Pois. ratio v(kN/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.0</td>
<td>38.0</td>
<td>19.5</td>
<td>1.0×10⁴</td>
<td>0.25</td>
</tr>
<tr>
<td>2</td>
<td>5.3</td>
<td>23.0</td>
<td>19.5</td>
<td>1.0×10⁴</td>
<td>0.25</td>
</tr>
<tr>
<td>3</td>
<td>7.2</td>
<td>20.0</td>
<td>19.5</td>
<td>1.0×10⁴</td>
<td>0.25</td>
</tr>
</tbody>
</table>

3.2 Results Based on Shear Strength Reduction Analysis

Based on the FLAC3D numerical analysis engine and its secondary development programs designed by the present authors [10], the FEM-SSR was performed. Besides, the associate flow rule (setting the dilation angle equal to the friction angle) was applied in an effort to limit the impacts of stress paths. After the cohesions and friction angles were reduced to 1.438 times, the slope is in a critical state. As the distribution location of shear strain rate (SSR) contour matched well with the shear strain increasement (SSI) contour in this case, it is enough to demonstrate one of them, and the SSI contour is shown in Fig. 1. In general, the anomaly of SSR or SSI contour indicates the development position of the CSS. And to judge from the contour, the system is in the edge that the upper and lower parts of CSS are not fully connected, but about to be linked together.

3.3 Comparisons among Five Different Stability Evaluation Methods

It is worth noting that a significant change of stress field has occurred before and after the FEM-SSR was performed. As shown in Fig. 1, there is an obvious difference between the two slip surfaces marked with No. 1 and No. 2 respectively. The No.1 slip surface coupled with FOS of 1.732 was obtained under the initial stress field (the initial weight stress field before performing FEM-SSR) by a global optimal searching algorithm with the FOS definition of Kulhawy (1969) [1]. Similarly, the No. 2 slip surface with corresponding FOS of 1.008 was acquired under the stress field yielded after FEM-SSR process by the same searching algorithm. Obviously, the difference between the positions of two slip surfaces is on the ground of the stress field’s change.

Compare with the FEM-SSR, there are two evidences can embodied the superiority of the searching algorithm (method 1). First, the location of No. 2 slip surface matched the SSI contour very well. Second, for stress field after the FEM-SSR process, the corresponding FOS of the No. 2 slip surface is only 1.008 which is extremely close to the theoretical value 1. These evidences show that the
searching algorithm based on the stress field is credible. So, due to the same algorithm, there is no reason to doubt the correctness of the results in condition of initial stress field. Besides, the corresponding FOS of No. 1 slip surface is 1.372 which is close to the ACADS’ recommended value of 1.39.

As a NP-Complete problem, although the slope stability evaluation can be performed by various global optimal algorithms, all these efforts are actually approximate methods. The searching algorithm applied in the present paper is the multi-level particle swarm optimization parallel algorithm developed by the present authors [10]. In essence, this method is based on the stress field which can be obtained by FEM or other numerical analysis engine. Additionally, this new searching algorithm has three major features: high-precision, high-speed, as well as global optimization. In this case, the slip surface is described as 31 nodes (with 62 degrees of freedom) which is detail enough for characterization the shape of slip surface, while the computational complexity is seriously increased. For most conventional slip surface optimization methods, the problem with 62 degrees of freedom is nothing but a nightmare. However, the new developed approach work well on it. In two aspects, it can clearly reveal the influences of stress fields before and after the FEM-SSR process. First, the location of CSS has a noticeable change along with the evolution of stress field. Second, for a same slip surface, the variation of FOS does not present a simple linear relationship with the strength reduction coefficient, which is, in essence, determined by the highly non-linear characteristics of the mechanical behaviour of the slope.

As a comparison, the Slope/E commercial program is also applied in this case. Several influential LEMs including the Janbu, Morgenstern-Price and Spencer were concerned, and the CSS is presented as Line No. 3 in Fig. 1. It is noteworthy that Slope/E can only perform a certain searching algorithm based on LEMs, and no matter what specific LEM method is selected, the acquired CSS is exactly the same one with circular shape. This indicates that the Slope/E utilizes a certain relatively simple strategy to determine the CSS under the constraint of circular shape. To determine a slip surface in two-dimensional problem in this simplified condition, only three parameters (three degrees of freedom) are required. On the contrary, the present authors’ method dedicated to achieve a precise description on slip surface with non-circular shape. In this case, there were 62 degrees of freedom are taken into account, and the complexity has hardly been the same as the mechanism of the Slope/E.
Table 2. The comparing of factor of safeties with different methods

<table>
<thead>
<tr>
<th>Method No.</th>
<th>Method</th>
<th>Method Classification</th>
<th>Initial stress field</th>
<th>The stress field after FEM-SSR processing (shear strength reducing 1.438 times)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>No. 1 surface</td>
<td>No. 2 surface</td>
</tr>
<tr>
<td>1</td>
<td>The multi-level particle</td>
<td>Combined</td>
<td>1.372</td>
<td>1.550</td>
</tr>
<tr>
<td></td>
<td>swarm optimization parallel</td>
<td>FEM &amp; LEM</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Janbu</td>
<td>LEM</td>
<td>1.261</td>
<td>1.280</td>
</tr>
<tr>
<td>3</td>
<td>Morgenstern-Price</td>
<td>LEM</td>
<td>1.439</td>
<td>1.365</td>
</tr>
<tr>
<td>4</td>
<td>Spencer</td>
<td>LEM</td>
<td>1.411</td>
<td>1.370</td>
</tr>
<tr>
<td>5</td>
<td>FLAC3D-SSR</td>
<td>FEM-SSR</td>
<td>FOS=1.438, and the critical slip surface (CSS) is present by the shear strain increasement contour</td>
<td></td>
</tr>
</tbody>
</table>

Note: No. 1 surface refers to the searching result of critical slip surface by method 1 in initial stress field.
No. 2 surface refers to the searching result of critical slip surface by method 1 in stress field after FEM-SSR processing.
No. 3 surface refers to the searching result of critical slip surface by the inherent mechanism of Slope/E, and the center coordinates X=35.46m, Z=41.49m, radius R= 17.19m.

It is noted that the three LEMs, Janbu, Morgenstern-Price and Spencer, themselves have the ability to analyze non-circular slip surface. Therefore, No. 1 and No. 2 surfaces which were located by the proposed new approach can also be processed by the Slope/E. As shown in Table 2, an interesting phenomenon is that among the three slip surfaces, no matter which LEM is concerned, the surface No. 3 can not yield the minimum FOS. In other words, the inherent searching algorithm of the Slope/E cannot obtain the exact CSS if non-circular surfaces are involved. The surface No. 1 for the Janbu method holds the minimum FOS, and the location is consistent with the one obtained by the method 1. The surface No. 2 for the Morgenstern-Price and Spencer methods holds the minimum FOS, in this time; the location matches the result of FEM-SSR (method 5). These mixed results are understandable; it is derived from the essential properties of the NP-Complete problems. It shows that, locating the CSS is a problem with considerable difficulty, and there is still a long way to advance the commercial software. Moreover, the searching algorithm with simplified strategy (such as Slope/E) might not get better results than the FEM-SSR, it is widely considered to have the powerful capability for automatic approaching the CSS.

It must be pointed out that, there is no concept of stress field in the theoretical framework of LEM. For the LEM, the affect on the FOS in various stress fields shown in the Table 2, is exactly the influence from the reduction of the strength parameters. For the same slip surface, using the same LEM, the FOS presents a simple linear relationship which coincided exactly with the reduction coefficient of strength parameters. Moreover, for the LEM, this feature of linear variation determines the location of the CSS will not change before and after the strength reduction. As mentioned above, this phenomenon is quite different from the condition of method 1 for the two stress fields which are obtained before and after the FEM-SSR process, respectively. In general, the difference is sourced from the distinctions of theoretical frameworks between the LEM and FEM.
3.4 Defects and Applicability of Shear Strength Reduction Analysis

Regardless considering the drift position of CSS before and after the FEM-SSR process and due to the numerical reasons, the FEM-SSR still has some difficulties to accurately capture the critical state. In some complex cases, the critical state capture is local, rather than overall, and it will lead to low estimation of the slope stability evaluation. The results of the FEM-SSR are highly depending on the choice of the numerical methods and software. Cheng et al. (2008) [12] pointed out, even with the same FEM theory background, the contours obtained from different commercial software have perceptible distinctions. In this case, the FOS obtained by the FEM-SSR is slightly higher than the one obtained from other approaches, and this phenomenon coincides with the researches of Zheng (2006) [9] and Cheng et al. (2007)[13].

Although the drift position of the CSS exists in the FEM-SSR, the convenience of this method makes it popular in the routine analysis. Moreover, compare with the complex and heavy duty searching algorithms, the general patterns and trends of the CSS are consistent. In some problems, it can be regarded as a simple and effective approaching. In terms of this case, the Morgenstern-Price and Spencer methods both indicate the No. 2 surface is critical, which matches the result of the FEM-SSR. And the significance of the reinforcement applications is that, although the CSS indicated by the SSI and SSR contours has not exactly reached the global minimum of FOS, the control positions of the CSS, such as shear opening in the toe and split trailing on the crest, have a basic agreement with the searching algorithms. Considering the reinforcement measures generally focusing on some critical areas which will take the lead of failure, the FEM-SSR still has a considerable practical value.

4 Conclusions

The research and results reported in this paper can be summarized as follows:

(1) The case study has proved that the equivalence between FEM-SSR and LEM does not exist, it is consistent with the theoretical conclusion of Zheng et al. (2006) [9] and Liu (2010)[10]. All the approaches, whether belong to the FEM LEM, or the integration of two types, have their respective premises and limitations and therefore the equivalent can hardly achieves. All of these methods will not completely replace each other.

(2) The FEM-SSR has inherent defects, which are featured by a notable position drift of CSS occurred after strength reduction. The FEM-SSR still has some difficulties to accurately capture the critical state, due to numerical reasons.

(3) Although there is no theoretical guarantee for the CSS obtained by FEM-SSR, the advantages of its convenience and concision can remedy its disadvantages of lacking precision, to some extent. Therefore, the analysis result of FEM-SSR is still an important reference for routine analysis.

(4) Based on the combined methods of FEM and LEM, a certain kind of advanced non-linear optimization technology such as swarm intelligence methods can be considered as a promising direction for further research of slope stability evaluation problem.

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References

IMPACT OF NATURAL FRACTURES TO THE STRESS FIELD BEFORE RE-FRACTURE IN THE RESERVOIRS

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The stress field is very complex after hydraulic fracturing in naturally fractured reservoirs. It is difficult to judge the fracture propagation direction during re-fracture. Impact of natural fractures to the stress field before re-fracture was simulated based on the effect of cracks and reservoir percolation. It was shown that the distraction along the minimum horizontal principal stress direction is much larger. The longer distance of two cracks, the smaller the disturbance is. The stress inversion region may be formed near the end of the natural fractures, when the distance between artificial fracture and natural fracture is shorter. The longer the natural fracture length, the greater the interference is. When the fracture length of artificial fracture exceeds a certain multiple of the natural fracture, the impact of a single natural fracture can be basically ignored. The stress field can meet the principle of superposition when there are two natural fractures in the reservoir.

1 Introduction

In fractured reservoirs, the stress distribution compared to the conventional reservoirs may be different affected by stress concentration effect of cracks and the multi-cracks interference. The interaction of the cracks depends on the geometric characteristics of theirs (including relative position, crack length, crack numbers, crack mechanics parameters, and so on) [1]. Therefore, it is necessary to study the stress field of the crack interference for accurately determining the crack propagation direction before re-fracturing. When the natural fractures exist at the extension direction of hydraulic fracture, it is studied by a large number of laboratory experiments and theoretical models about the hydraulic fracture propagation [2-10]. In fact, even through the natural fractures do not intersect at the extension direction of hydraulic fracture, the propagation direction of hydraulic fracture may be changed. Generally, the direction of artificial fracture propagation is affected by the stress distribution. In this paper, based on the percolation effect of cracks and matrix, it was studied of the stress distribution before re-fracture in fractured reservoirs. A guideline was provided in this paper about the further judgement of the crack re-direction in fractured reservoirs.

2 Basic Model

For the approximation of flow pressure distribution between two-phase flow and single-phase one, single-phase flow of incompressible fluid was studied based on fluid-solid coupling theory. The basic model is shown in Fig.1.
Basic assumption:
(1) Rock in plane strain state;
(2) Linear elastic rock deformation, not fracturing;
(3) Plane two-dimensional percolation, isothermal flow.

2.1 Reservoir Flow Equation

Before re-fracture in fractured reservoirs, the fluid flow equations include percolation equations of matrix and natural fracture. The mathematical models were established as follows:

Percolation equation of matrix system

\[ \nabla \left( \frac{1}{\mu} K \nabla p \right) - \tau (p - p_f) + q = \phi C L \frac{\partial \varphi}{\partial t} + \frac{\partial \varphi}{\partial t} \]

Percolation equation of natural fracture

\[ \nabla \left( \frac{1}{\mu} K \nabla p \right) + \tau (p - p_f) + q = \phi C L \frac{\partial \varphi}{\partial t} \]

Where, \( \mu \) is fluid viscosity, \( \text{Pa} \cdot \text{s} \); \( K \) is absolute permeability of the reservoir, \( \text{m}^2 \); \( \tau \) is fluid exchange coefficient in the deformation medium reservoir and fracture by volume; \( p \) is formation pressure, \( \text{Pa} \); \( p_f \) is pressure in fracture, \( \text{Pa} \); \( q \) is terms of sources and sinks by unit volume, \( \text{kg/}(\text{m}^2 \cdot \text{s}) \); \( \phi \) is porosity; \( C_L \) is compressibility coefficient, \( 1/\text{Pa} \); and \( \varphi \) is volumetric strain of rock frame.

2.2 Control Equation of Deformation Field

According to the force balance principle and the Terzaghi effective stress formula, the deformation field balance equations can be drawn. They are the basic equations for solving porous media deformation.

\[ \frac{\partial \sigma_x}{\partial x} + \frac{\partial \tau_{xy}}{\partial y} + f_x - \alpha \frac{\partial \varphi}{\partial x} = 0 \]

\[ \frac{\partial \sigma_y}{\partial y} + \frac{\partial \tau_{xy}}{\partial x} + f_y - \alpha \frac{\partial \varphi}{\partial y} = 0 \]

Where, \( \alpha \) is Biot coefficient.
To solve the above equations, some auxiliary equations are needed. Firstly, based on small deformation theory, the relationship between the strain components and displacement components can be described by the following geometric equation,

\[
\begin{pmatrix}
\varepsilon_x \\
\varepsilon_y \\
\gamma_{xy}
\end{pmatrix} = \begin{pmatrix}
\frac{\partial u}{\partial x} & \frac{\partial u}{\partial y} & 0 \\
\frac{\partial v}{\partial x} & \frac{\partial v}{\partial y} & 0 \\
\frac{\partial u}{\partial y} + \frac{\partial v}{\partial x} & \frac{\partial v}{\partial y} & 0
\end{pmatrix} \begin{pmatrix}
\frac{\partial}{\partial x} & 0 & 0 \\
0 & \frac{\partial}{\partial y} & 0 \\
0 & 0 & \frac{\partial}{\partial x}, \frac{\partial}{\partial y}
\end{pmatrix}
\begin{pmatrix}
u \\
u
\end{pmatrix}
\]

where, \( \varepsilon_x, \varepsilon_y \) are the strain components, \( \gamma_{xy} \) is the shear strain component, \( u \) and \( v \) are the displacement components in the \( x \) and \( y \) directions, respectively.

Secondly, the relationship between stress and strain can be shown by generalized Hook's law for linear elastic solid,

\[
\begin{align*}
\varepsilon_x &= \frac{1}{E} (\sigma_x - \nu \sigma_y) \\
\varepsilon_y &= \frac{1}{E} (\sigma_y - \nu \sigma_x) \\
\gamma_{xy} &= \frac{2}{E(1+\nu)} \tau_{xy}
\end{align*}
\]

where, \( E \) is elastic modulus of rock matrix, GPa, and \( \nu \) is Poisson ratio.

3 Case Study

Elastic modulus of rock matrix is 2.4GPa, Poisson ratio is 0.24, half-fracture length of artificial fracture is \( a = 10m \), half-fracture length of natural fracture is \( l = 6m \), fracture length and width ratio is much larger than 1, permeability of rock matrix is \( K = 15 \times 10^{-3} \mu m^2 \), porosity is \( \phi = 0.2 \), fluid viscosity is \( \mu = 3mPa \cdot s \), the initial maximum horizontal principal stress is 30MPa, the initial minimum horizontal principal stress is 20MPa, net pressure of artificial fracture surface before re-fracture is 15MPa, initial reservoir pressure is 10MPa. It can be simulated about the interference of natural cracks to stress under the quasi-static conditions before re-fracturing.

3.1 Influence of Inclination Angle about Natural Fracture

Assuming natural fracture endpoint \( c \) changes at \( x = a \), the stress field (compressive stress is negative) under different inclination angle of the natural fracture can be analyzed, as shown in Fig. 2. The greater the inclination angle, the smaller influence the natural fracture to the effective stress is; The stress field is disturbed much largely along the original minimum principal stress direction in the vicinity of the natural fracture and it can be close to the original stress field of non natural fracture at infinity. Specially, reverse stress region may be produced due to rapidly increase of \( \sigma_y \) in a small area after the end of the natural fracture point \( d \), as shown in Fig.2 (a) and (b). These results can provide a reference for judging the direction of crack propagation in micro-fractured reservoirs when re-fracturing.
3.2 Influence of Horizontal Space between Artificial Fracture and Natural Fracture

Assuming natural fracture endpoint c changes at $y=1$, the numerical simulation results were shown in Fig. 3. The effective stress distribution has the same tendency near the natural fracture, if the location of natural fracture is changed in the horizontal direction. The changes of effective stress at the middle of the natural fracture are much larger, not considering the singular problem of the crack tip. Similarly, the disturbance in y direction is greater than in x one.

3.3 Influence of Fracture Length

Considering the position of the point c and inclination angle of natural fracture as constant, the results are shown in Fig. 4. In the figure, the fracture length ratios between artificial fracture and natural fracture are 10:1, 5:2 and 1:1, respectively. Apparently, the longer the natural fracture length, the greater the interference is, compared with the force field of non natural fracture. When the fracture length ratio is equal to 10:1, the disturbance of natural fracture can be basically ignored. Similarly, for any position of natural fracture, when the fracture length of artificial fracture exceeds a certain multiple of the natural fracture, the impact of a single natural fracture can be basically ignored.
3.4 Influence of Double Natural Fractures

When inclination angle of the natural fracture is 30°, for two natural fractures upper and lower symmetrically on the x-axis, the effective stress variation is shown in Fig.5. The stress field can meet the principle of superposition when there are two natural fractures in the reservoir. Therefore, as a result of the additive effect, the force field can be influenced by multiple short cracks.

4 Conclusions

In this paper, the influence of natural fractures to the stress field before re-fracture was simulated based on the effect of cracks and reservoir percolation. The conclusions are shown as follows,

(1) The distraction due to natural fracture along the minimum horizontal principal stress direction is much larger. When the distance of two cracks is much longer, the disturbance is very small.

(2) The stress inversion region may be produced near the end of the natural fractures, when the distance between artificial fracture and natural fracture is shorter.

(3) Compared with non natural fracture, the longer the natural fracture length, the greater the interference is. When the fracture length of artificial fracture exceeds a certain multiple of the natural fracture, the impact of a single natural fracture can be basically ignored.

(4) The stress field can meet the principle of superposition when there are two natural fractures in the reservoir.
References

THE EFFECTS OF GAS PRESSURE ON GAS INFILTRATION CHARACTERISTICS IN COAL

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In order to study the characteristics of gas infiltration in coal body surrounded by rigid rock under the conditions of different gas pressure, a self-developed coal-gas sorption/desorption simulation device is used to establish a physical coal-gas infiltration model. In this experiment, two types of gases (N₂ & CO₂) are used respectively to simulate gas penetrating through the tightly contained coal body, and gas pressure, total stress and infiltration velocity are monitored. The results show that the relationship between gas infiltration velocity in gas-contained coal and gas pressure is fitted by second-order polynomial. The results also show that, under the conditions of same temperature and same pressure, the infiltration velocity of N₂ is higher than that of CO₂. The experimental study is important for coal-gas hazard prevention and CBM exploration in coal mine.

Coal seam gases are harmful gases which affect coal mine safety and produce atmospheric greenhouse effect, and they are clean energy at the same time[1]. Most of coal seams in our country have the characteristics of high geostress, high gas pressure and low permeability. Along with the increase of mining depth and speed, coal gas dynamic phenomenon become more serious, and gas pressure is one of the important factors. Gas pressure has complex influence to permeability characteristics of coal, because coal gases seepage in the pores of coal body is accompanied by gas diffusion, adsorption and desorption[2-7]. Meanwhile, it goes along with the deformation behavior of coal body and mechanical variation. When coal gas is in the process of adsorbing in coal body surrounded by rigid rock, the total stress and the effective stress increases, and coal matrix deformation causes pore expansion and the fractures in coal has the trend of closeing, and the permeability of coal body decreases as a result[8].

Gas pressure is one of many factors affecting coal gas permeability, while few researches of gas pressure’s effect on permeability of coal are reported. YUAN M[9] studied the relationship between pressure on both ends of coal sample and the permeability of coal under the condition of constant effective stress, temperature and air pressure. YIN G Z[10] developed an three axis permeation apparatus and studied the relationship between pressure and coal permeability under the condition of fixed axis pressure and confining pressure. This paper studied the relationship between stable seepage pressure and coal gas permeability in coal body surround by rigid rock. Research results are expected to benefit gas extraction and gas geologic disasters prevention in coal mine.

1 Experimental equipment and samples

1.1 Experimental equipment

A self-developed sorption/desorption device is used to simulate the process of coal gas infiltration. The test system consists of coal sample installing system, compression system, gas injection system and data acquisition system[11]. As is shown in fig.1:
1.2 Coal samples and gas samples

Coal samples are taken from No.13 coal seam of Pan-three coal mine in Huainan coal field. Sampling site is 728 meters deep, the type of the coal sample is long-flame coal, the density of the coal sample is 1.33 g/cm³. Exogenous cracks in the coal are well developed, and therefore this coal sample belongs to tectonic coal \[12\]. To quantify broken degree of coal sample, random 1.215 Kg coal samples were taken and screened. The result is shown in table 1:

<table>
<thead>
<tr>
<th>Grain size of coal sample (mm)</th>
<th>Mass percentage (%)</th>
<th>Mass of coal (kg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; 0.18</td>
<td>10.70</td>
<td>0.13</td>
</tr>
<tr>
<td>0.18~0.25</td>
<td>5.76</td>
<td>0.07</td>
</tr>
<tr>
<td>0.25~0.425</td>
<td>5.35</td>
<td>0.065</td>
</tr>
<tr>
<td>0.425~1.18</td>
<td>19.75</td>
<td>0.24</td>
</tr>
<tr>
<td>1.18~3.35</td>
<td>29.22</td>
<td>0.355</td>
</tr>
<tr>
<td>&gt; 3.35</td>
<td>29.22</td>
<td>0.355</td>
</tr>
</tbody>
</table>

From table 1, it is known that mass percentage of coal samples below size 0.45 mm achieve 21.81%. The coal samples belong to tectonic coal because mass percentage of coal samples whose particle size is below 0.5 mm surpass 20\% \[13\].

Physical adsorption and desorption of coal gas are reversible \[14,16\], and N₂ & CO₂ are composition of coal gas. In this experiment, N₂ and CO₂ are used respectively to test coal gas permeability.

2 Experimental method

Firstly, 60 Kg coal samples whose particle size are below 30 mm are taken into sample installing system, and two total stress sensors (No.1 & No.2) are placed in coal body in order during the process of sample installing. Secondly, the cover is closed and coal samples are compressed to high density under the action of hydraulic system whose pressure is up to 25 Mpa. The pressure is kept 240 h until stability. Six pressure sensors and six temperature sensor are installed in drilled holes on the cylinder wall of the sample installing system, and cylinder sealing quality is checked.

Two gases of N₂ and CO₂ are used to test coal permeability respectively. First, gas with fixed inlet pressure is to penetrate the coal sample. When coal gas infiltration state reach dynamic equilibrium and the pressure of all measure points is stable, gas permeability velocity can be recorded, and the gas spilled out of coal through pipe can be collected. Then, the next infiltration test can be done after
changing gas inlet pressure.

Infiltration test ends when coal gas pressure gradually drops to atmospheric pressure. Coal samples need not to be removed and N<sub>2</sub> adsorption test can be done under the condition of constant entrance pressure.

The arrangement of total stress sensors and pressure sensors is shown in fig.2:

```
<table>
<thead>
<tr>
<th>No.1 No.2</th>
<th>No.3 No.4</th>
<th>No.5 No.6</th>
</tr>
</thead>
<tbody>
<tr>
<td>gas pressure sensors</td>
<td>130</td>
<td>130</td>
</tr>
<tr>
<td>piston</td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>signal lines</td>
<td>signal lines</td>
<td>signal lines</td>
</tr>
</tbody>
</table>
```

![Fig.2 Sketch of sensors arrangement](image)

3 Test results and analysis

3.1 Coal gas infiltration features under the conditions of different pressure

N<sub>2</sub> and CO<sub>2</sub> are used respectively to penetrate coal in container, the pressure is 0.2 MPa, 0.4 MPa, 0.6 MPa, 0.8 MPa, 1.0 MPa, 1.2 Mpa and 1.4 MPa respectively. The curves of infiltration velosity with gas pressure are shown in figure 3:

\[
y = -9.5262x^2 + 56.941x - 8.9271 \\
R^2 = 0.996
\]

\[
y = -9.1591x^2 + 139.17x - 8.1929 \\
R^2 = 0.9999
\]

![Fig.3 Comparison of infiltration velocity between N<sub>2</sub> and CO<sub>2</sub> under different gas pressure](image)

In fig.3, the functional relationship between gas infiltration velocity in coal body and gas pressure is fitted by a second-order polynomial on condition that coal temperature keep constant. Analysis shows that coal gas diffusion and absorption actions occur during the process of coal gas infiltrating with a certain pressure, coal matrix swelling caused by adsorption leads to that seepage channels become narrow. If there were no adsorption swelling effect in coal matrix, there would be no variation in fracture channels, and the relationship between infiltration velocity and gas pressure would be linear. it is just because the adsorption swelling effect leads to changes of infiltration chanel width. As a result, infiltration velocity changes with gas pressure according to second-order polynomial.

It can also be seen from data in fig.3 that infiltration velocity of N<sub>2</sub> in coal body is far higher than that of CO<sub>2</sub>, and start-up pressure of N<sub>2</sub> to penetrate a certain length coal cylinder is lower than that of CO<sub>2</sub>. It can be derived that infiltration velocity of N<sub>2</sub> reach its peak of 520.47 cm<sup>3</sup>/s when pressue arrives at 7.60 MPa, while that of CO<sub>2</sub> reach its peak of 76.16 cm<sup>3</sup>/s when pressue arrive at 2.99 MPa through derivativing the two fitted equations respectively. The root cause of producing obvious
difference of infiltration ability and start-up pressure between the two gases is their different physical and chemical properties. Their physical and chemical properties are shown in table 2:

Table 2 Physical and chemical parameters’ effects on sorption of CO$_2$, CH$_4$ and N$_2$ [17]

<table>
<thead>
<tr>
<th>Physical and chemical parameters</th>
<th>N$_2$</th>
<th>CH$_4$</th>
<th>CO$_2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Boiling temperature (°C)</td>
<td>-195.81</td>
<td>-161.49</td>
<td>-78.48</td>
</tr>
<tr>
<td>Critical temperature (°C)</td>
<td>-147.0</td>
<td>-82.01</td>
<td>31.04</td>
</tr>
<tr>
<td>Critical pressure (MPa)</td>
<td>3.398</td>
<td>4.641</td>
<td>7.386</td>
</tr>
<tr>
<td>Critical density (kg/m$^3$)</td>
<td>314</td>
<td>426</td>
<td>466</td>
</tr>
<tr>
<td>Ionic potential (eV)</td>
<td>13.0</td>
<td>13.79</td>
<td>15.6</td>
</tr>
<tr>
<td>Effective diameter (nm)</td>
<td>0.374</td>
<td>0.414</td>
<td>0.456</td>
</tr>
<tr>
<td>Adsorptive capacity</td>
<td>low</td>
<td></td>
<td>high</td>
</tr>
</tbody>
</table>

Data in table 2 show that different physical and chemical properties of the two gases caused the quantity of CO$_2$ odsorbed in coal are more than that of N$_2$ under the same conditions of temperature and pressure, and adsorption swelling effect created by CO$_2$ infiltration is more obvious than that of N$_2$. Therefore, the obvious permeability difference between the two gases is caused by adsorption swelling effect.

3.2 Swelling effect of coal gas adsorption

To further illustrate the adsorption swelling effect, N$_2$ is used to test adsorption features of coal body sealed in sample installing cylinder under the condition of isothermal and constant pressure. Total stress curves of No.1 sensor and No.2 sensor during the process of absoption at 0.2MPa, 30°C are shown in fig. 4:

![Fig. 4 Total stress curves of No.1 sensor and No.2 sensor during the process of absoption at 0.2MPa, 30°C](image)

It can be seen from fig. 4 that total stress and effective stress change with time according to second-order polynomial rule respectively under condition of constant inlet pressure, constant coal temperature and rigid surrounding rock. Consequently, coal matrix swell constantly. No. 1 and No. 2 total stress sensors are in high stress area and low stress area of coal body respectively, adsorption swelling effect at high stress area is more obvious than that of low stress area. The main reason is that high stress area has high density and high adsorption capacity. As a result, coal matrix in high stress...
area has large swelling effect, and total stress and effective stress in high stress area are higher according to hooke's law.

It can also be seen from fig.4 that coal gas adsorption equilibrium time at positions of No. 1 and No. 2 total stress sensors are 1375 min and 1000 min respectively through differentiating the above equations, it means that equilibrium time of high stress area is longer. The reason is that compression degree of pores in high stress area is higher and porous diameter is smaller. As a result, permeability and diffusion velocity decreases, and the time of adsorption equilibrium increases.

4 Conclusions
According to data analysis of infiltration and adsorption test, the following conclusions can be achieved:

(1) Coal gas infiltration velocity in rigid surrounding rock changes with gas pressure according to second-order polynomial rule.

(2) The infiltration velocity of N2 in coal body surrounded by rigid rock is higher than that of CO2 under the condition of isothermal and constant pressure.

(3) Total stress and effective stress of coal enclosed by rigid rock change with adsorption time according to second-order polynomial rule on condition that gas inlet pressure and coal body temperature stay the same.

Reference:


THE RESEARCH AND APPLICATION OF JOINTED CONSTITUTIVE MODEL OF FLAC SIMULATION FOR THE LAYERED ROCK SLOPE IN THE OPEN-PIT

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(College of Mining Engineering, University of Science and Technology LiaoNing, Anshan 114051 China)

The failure modes of the layered structure of the rock slope is planar slip and toppling, the reason of this kind of failure is deformation and strength anisotropy. In this paper, we conduct the FLAC numerical simulation to the bedding rock slop of the open-pit with the the jointed constitutive model. The dip angle of the slop with the plasticity glide band and the dip angle of the layer is generally consistent. The FLAC jointed constitutive model can provide an effective means for the simulation of layerde rock masses.

1. Introduction

The stability of the slope of open-pit is the significant technical issues in the mine safely and efficiently. The slope steepness increases with the strip mining. The stability of slope decrease and the risk of landslide increase[1,2]. The rockmass was cut into macroscopic discontinuous geologic structure by the various structural features (such as faults, joints, bedding, schistosity, fracture zone, weak interlayer structural plane). Rock is more complex than any engineering material ever known so far because of its heterogeneity, discontinuity and anisotropy. Rock mechanics workers keep engaging in the arduous exploration. Limiting equilibrium plays a very important part in assessment of slope stability, but the process has fundamental theoretical defect because of the assumes of rigid model. Numerical simulation analysis showing strong vitality with the rapid advances of computer technology, however, the process is still immature in applications of engineering. This method still require a lot of in-depth study to gradually improve. The layered body of rock is a common medium in the assessment of slope stability. It has a certain practical significance to analysis and investigate the deformation and failure mechanism.

2. Jointed constitutive model and FLAC model

Jointed constitutive model (Ubiquitous) is the anisotropic plasticity model provided by the FLAC. It consists of a weak surface included in the special direction of the Mohr - Coulomb structure, it can be used to simulate the layered rock mass destruction.

Figure 1 Overall coordinate system direction along the $\theta$ angle in weakness

The yield may occur in the body of the model or on the weak surface. According to the difference of stress state, the weak surface direction and material properties of the model body and the weak
surface, the yield also probably occur in two parts simultaneously. It can be seen from Figure 1, the weak surface exists in the Mohr-Coulomb structure under the overall coordinate system (x, y) and the local coordinate system (x', y').

The implementation methods of orthogonal-jointed constitutive model in FLAC is to distinguish the structural failure firstly. At the same time, it is apply to related plasticity correction in Mohr-Coulomb model of the FLAC. Secondly, it analyze the damage on the weak surface from the update stress and calibrate these stress more precisely. The failure criterion of Weak surface exists in the Local form with Mohr-Coulomb yield condition which include the the path of the tensile stress. And the criterion does not associated with the local shear flow rule but associated with the flow rule of the local tensile stress. The failure criterion of Weak surface can be showed in the figure 2.

Mohr-Coulomb failure criterion is defined as \( f^s = 0 \), the partial failure envelope from point A to point B is defined as:

\[
f^s = -\tau - \sigma_{22}' \tan \phi_j + C_j
\]

The criteria of tensile stress damage is defined as: \( f^t = 0 \), the partial failure envelope from point B to point C is defined as:

\[
f^t = \sigma_j' - \sigma_{22}'
\]

In the formula, \( \phi_j, C_j, \sigma_j' \)—were weak surface friction angle, cohesion and tensile strength.

The shear potential function \( g^s \) and Tension potential function \( g^t \) corresponding to the non-associated flow rule, dilatancy angle corresponding to the associated flow rule. They are expressed as:

\[
\begin{align*}
g^s &= -\tau - \sigma_{22}' \tan \psi_j \\
g^t &= -\sigma_{22}'
\end{align*}
\]

The function of the flow rule near the boundary is in the form of:

\[
h = \tau - \tau_j^p - \alpha_j^p (\sigma_{22}' - \sigma_j')
\]

Among the formula, \( \tau_j^p \) and \( \alpha_j^p \) are two constants.

\[
\begin{align*}
\tau_j^p &= C_j - \tan \phi_j \sigma_j' \\
\alpha_j^p &= \sqrt{1 + \tan \phi_j} - \tan \phi_j
\end{align*}
\]

The flow rule of the shear failure in the plane is:
\[ \Delta e''_{11} = \lambda' \frac{\partial g'}{\partial \sigma'_{11}} \quad \Delta e''_{22} = \lambda' \frac{\partial g'}{\partial \sigma'_{22}} \]
\[ \Delta e''_{33} = \lambda' \frac{\partial g'}{\partial \sigma'_{33}} \quad \Delta \gamma'' = \lambda' \frac{\partial g'}{\partial \tau} \]

In the formula, \( P \) expresses the plastic part on the weak surface which is associated with the destruction, \( \lambda' \) is the unknown parameter, use the formula \( g'' = -\tau - \sigma'_{22} \tan \psi' \). Carry out partial differential:

\[ \Delta e''_{11} = 0 \quad \Delta e''_{22} = -\lambda' \tan \psi' \]
\[ \Delta e''_{33} = 0 \quad \Delta \gamma'' = -\lambda' \]

We can get the total stress correction of the shear failure on the weak surface from the local stress correction, and figure out the new stress state of this step by these stress correction.

The flow rule of pull stress fracture on the weak surface as follows:

\[ \Delta e''_{11} = \lambda' \frac{\partial g'}{\partial \sigma'_{11}} \quad \Delta e''_{22} = \lambda' \frac{\partial g'}{\partial \sigma'_{22}} \]
\[ \Delta e''_{33} = \lambda' \frac{\partial g'}{\partial \sigma'_{33}} \quad \Delta \gamma'' = \lambda' \frac{\partial g'}{\partial \tau} \]

In the above formula, \( \lambda' \) is the undetermined parameter, and the following formulae are the deformation of partial differential by use the formula of \( g' = -\sigma'_{22} \):

\[ \Delta e''_{11} = 0 \quad \Delta e''_{22} = -\lambda' \]
\[ \Delta e''_{33} = 0 \quad \Delta \gamma'' = 0 \]

We can figure out the modified value of destructive tensile stress in weak face as well.

The slope of layered rock mass slips along bedding plane is the main features of its deformation and failure as well as manifestation of the anisotropy intensity. Section of ubiquitous of FLAC could afford a simulative means of layered rock mass.

3. Applicable analysis of actual project.

The iron ore footwall slope rock mass contains mainly chlorite-gneiss in Anshan Iron and Steel, the plate-like chlorite-gneiss comprise bedding structure of the rock slope having the value of 42°~50° of angle of rock layer, and it plays a controlling role with the deformation and failure of slope.

By using C profile slope shape designed by mining scheme, the simplified calculation model of FLAC ubiquitous was established. The slope top height mark is 280m. The lowest mining level is 72m. The final altitude of slope is 280m and the slope angle is 40°. Firstly, the in-situ stress is calculated. Secondly, based on the benching cut, the simulation mining of 11 steps is carried out. The height of benching cut is 264m, 240m, 216m, 192m, 168m, 158m, 144m, 120m, 99m, 96m and 72m respectively. The model calculation parameters of rockmass is density 2620kg/m^3, shear modulus 5.71 GPa, volume modulus 6.67 GPa, cohesion 500 KPa, friction angle 30°. The dip of structural plane is 45°. The parameters of structural plane is cohesion 10 KPa, the friction Angle 25°.
Beginning of the simulated mining, every step of the mining simulation only express the performance of instantaneous plastic flow of the rock near the slope along the rock layers, then the elastic recovery. In fact, this phenomenon should be the null of elements at once of simulation. Generally, this does not appear this result in practice. The rock is gradually stripped in the process of mining production. It can be seen from the seventh step simulated mining (144 m level, in Figure 3), the step slope toe at 168m and the slop from 168m to 144m appear two plastic slip bands along the rock layers of inclination about 45°. Slip bands inclination basically consistent with the inclination layers. With the continuing downward mining, two plastic slip bands along the level do not extend and the extension is less than 120 meters. At the end of the simulated mining (in Figure 4), there appears a new plastic slip band along the rock layers, the slip band extends to about the lowest mining level.

Fig.3 plastic zone of the seventh step mining simulation
Fig.4 plastic zone of the end of Simulated mining
Fig.5 the maximum principal stress cloud picture of the end of the simulated mining
Fig.6 the minimum principal stress cloud picture of the end of the simulated mining
Fig. 7 the horizontal displacement cloud picture of the end of the simulated mining Fig. 8 the vertical displacement cloud picture of the end of the simulated mining

4. Conclusion

The occurrence, intensity and other parameters play a major role in the deformation and failure of the layered rock slope. Particularly, when dip of structural surface is consistent with the slope, the slope stability will be worse. Slipping along layers is the main features of deformation and failure of slope. It is also the performance of deformation and strength anisotropy. Projects simulations showed that, jointed constitutive model of FLAC can provide an effective way for simulation layered rock masses. Considering the thick of structural, the simulation of the deformation anisotropy needs to be studied further.

Reference

SAFE DISTANCE AND PERTURBATION PRESSURE DETERMINATION METHOD OF TUNNEL PASSING THROUGH BENEATH THE LANDSLIDE

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While highway or railway has been built in mountainous areas, it would inevitably encounter landslide and often be selected the option that the tunnel passing through beneath the landslide. More and more attentions were paid to tunnel disease in landslide field by the engineering community, but there is no recognized solution in theory. This paper focuses on tunnel passing through beneath landslide, emphasizing that the tunnel-landslide should be regarded as an overall interaction. It is established the concept of 'tunnel-landslide' system, which has an extremely important role in correct understanding the relationships of tunnel excavation and landslide peristaltic but also analyze the deformation mechanism and make engineering control measures. Studies show that when the tunnel passing through beneath landslide, the safety distance (H) from tunnel vault to the sliding surface is related to the factors of tunneling radius, sturdy coefficient of slippery bed, thickness of the slip mass, angle of internal friction, and so on. Derived a formula of tunnel-landslide perturbation range, by the formula, safe distance (H) from tunnel vault to the sliding surface is computable, and verified by numerical simulation, shows that this method can obtain a satisfactory results. Analyzed the force of tunnel-landslide system, derived a formula of vertical perturbation pressure when tunnel-landslide interaction. And pressure acting on tunnel support structure is the sum of normal state (without consider the landslide ) ground pressure and tunnel-landslide vertical perturbation pressure.

1 Introduction

In road route selection process, sometimes in order to ensure the overall layout of the line or change line is uneconomic and often be selected the tunnel option passing through beneath the landslide [1]. How to determine the safe distance from the sliding surface to tunnel vault in different level of surrounding rock under different geological conditions is very important when tunnel passing through beneath the landslide. At present, little research about determination the disturbance range of tunnel-landslide and whether or not the tunnel support strength is enough to resist the perturbation pressure at home and abroad. Research tunnel-landslide disturbance range and the forces act on tunnel if the tunnel within the range plays an important role for enhancing the concept of tunnel-landslide system, and providing a theoretical basis for the actual project to take reasonable protective measures. Therefore, the research of safe distance of tunnel beneath the landslide has a very important theoretical and engineering significance.
2 Concept of tunnel-landslide system

Some scholars have begun to concern the engineering problem generated by tunnel excavation in landslide field, they study tunnel deformation combined with landslide hazard and achieved certain results [2-9]. Deformation control theory does not have a recognized solution in the process of engineering construction and operation of tunnel passing through landslides and other geological diseases. The author believes that the tunnel deformation with close interaction between slope deformation in the process of tunnel construction and operation. To avoid the trade-off, established the concept of 'tunnel-landslide' system. Intended to alert colleagues to more fundamentally understand the interaction and deformation mechanism of the system, and provide a more accurate theoretical basis in practical projects to take reasonable precautions.

Tunnel-landslide system described in this article refers to a variety of collective in which tunnel construction and operation within landslide fields and its surrounding areas which have a certain influence. The main intent to establish the system is to take into account both the tunnel and landslide deformation characteristics, but also consider the interaction between them as a system comprehensive analyze its deformation mechanism and control technology.

3 Tunneling disturbance range of surrounding rock

Excavation will cause adjustment of surrounding rock stress state within a certain range of tunnel periphery, namely stress redistribution. Then the tunnel periphery formed loose zone, plastic zone and elastic zone, due to the emergence of loose zone and plastic zone will allow the tunnel in an unstable state, therefore, the range of the plastic zone has been paid more concerned about the object. Consider the worst case, the tunnel excavation failed to exert a supporting structure, the plastic zone can reach to the largest degree (Rpmax). Practice has proved that in circular chamber periphery will occur circumferential destruction when the lateral pressure coefficient $\lambda = 1$; But when the lateral pressure coefficient $\lambda < 1$, on both sides of the middle of the surrounding rock will occur 'broken wedge', the broken wedge in plastic zone at the highest stress concentration factor, that is, stress reduction and strength loss most serious. In landslide field, tunnel with shallow depth, and usually the lateral pressure coefficient $\lambda < 1$, have the characteristics of plastic slip damage; For the case of $\lambda \neq 1$ can also be used for the case of $\lambda = 1$ slip-line equations to calculate the length of the broken wedge. Therefore, we could derive the range of the plastic zone in accordance with the slip-line theory [10].

According to Mohr's circle theory, slip-line in the plastic zone have $45^\circ + \phi/2$ angle with minimum principal stress trace direction, i.e the axis to $\alpha$ angle. When the axisymmetric situation $\alpha = 45^\circ + \phi/2$, and the calculating diagram shown in Figure 1.
As shown in the figure, when coordinate changes in \( \Delta \theta \), the radial changes \( dr \), \( \theta \) by \( \rho \) to \( \theta \), \( r \) from \( r_0 \) to \( r \). Therefore,

\[
 dr = r d\theta \cot\left(45^\circ + \frac{\varphi}{2}\right)
\]

(1)

\[
 \int_{r_0}^{r} \frac{dr}{r} = \cot\alpha \int_{\rho}^{\theta} d\theta
\]

(2)

\[
 r = r_0 e^{(\theta - \rho)\cot\left(45^\circ + \frac{\varphi}{2}\right)}
\]

(3)

Where: \( \rho \)——Initial broken angle;

\( \varphi \)——Internal friction angle of surrounding rock.

Another group of slip-line is:

\[
 r = r_0 e^{-(\theta - \rho)\cot\left(45^\circ + \frac{\varphi}{2}\right)}
\]

(4)

Predictably, slip-line in the plastic zone is a composition on staggered helix.

According to the theory of slip-line above, we can calculate the maximum radius of plastic zone when tunnel excavation:

\[
 R_{p_{\text{max}}} = r_0 e^{(\theta - \rho)\cot\left(45^\circ + \frac{\varphi}{2}\right)}
\]

(5)

4 Perturbation range of landslide peristalsis after tunnel excavation

4.1 Computational model and basic assumptions

When the landslide beneath un-tunneling, the slip mass will slide down along the sliding zone, with very small disturbance range on the slippery bed in sliding process, generally not considered. However, when tunnel excavation beneath the landslide, on the one hand, a large disturbance on the surrounding rock during tunnel excavation will cause a loose zone, plastic zone and elastic zone in tunnel periphery, pressure acting on the supporting structure considered separately after tunnel excavation has a more detailed theoretical solution; on the other hand, for tunnel-landslide system, the formation of a large free face under the landslide after the tunnel was excavated will lead the loose slip mass above the vault collapse directly to the tunnel, produce a similar roof falling disease if the tunnel vault close to sliding...
surface. Tunnel disturbance for the upper slope gradually decrease with the increase of distance from the tunnel vault to sliding surface.

Tunnel excavation changed the stress state within a certain range of the landslide, we believe that the exist of tunnel has influence on the landslide as long as stress state of the landslide changed in the process of stress redistribution. Subsequently, the landslide motility increased, resulting in extrusion of the landslide to the tunnel free face. In any part of the peristaltic squeeze zone are likely to produce great squeeze pressure and in the opposite direction by equal opposite force in equilibrium (Figure 2). Suppose that in the range of peristaltic squeeze zone, squeeze pressure is uniformly distributed, with size q. Besides, a plastic ring was formed when tunnel excavation, surrounding rock stability within plastic ring is low, which is easy to collapse. Geo-materials local in plastic ring straight up until the surface subject to the greatest impact, which has the trend of move to the tunnel free face direction and driven on both sides of the soil mass destruction. According to Mohr-Coulomb’ s theory, geo-materials damaged along with the maximum principal stress direction at $45^\circ - \phi/2$ angle. Due to the soil subject to clamping action on both sides, the range of the landslide peristaltic squeeze zone are more down to the smaller, and ultimately formed a groove-shape extrusion zone. Suppose the boundary of the groove-shape extrusion zone can be represent by quadratic curve, and the curve height y can be deduced according to the balance of forces.

As shown in Figure 2, according to the geometric relationship, span of the quadratic curve (D) is:

$$D = \frac{2R_{p_{max}}}{\cos \beta} + Ah$$

(6)

$$A = \sin(45^\circ - \phi') \left[ \frac{1}{\sin(45^\circ + \beta + \phi')} + \frac{1}{\sin(45^\circ - \beta + \phi')} \right]$$

Where: factor

- $h$—Thickness of slip mass above the vault;
- $\phi'$—Internal friction angle of slip mass
- $\beta$—Angle of the line connecting the two intersection point of fracture surface on the slip surface with the horizontal.

![Fig. 2 Analysis model of tunnel-landslide system](image1)

![Fig. 3 Sinking arch calculation diagram](image2)
4.2. Sunken arch axis equation

Assume after the tunnel excavation, the slip mass squeezing toward the excavation free surface and formed a groove-shape sinking arch, its border can be represent by quadratic curve. Arbitrarily take a point M(x, y) on the arch, according to the properties of geo-materials, the arch axis can only withstand compression pressure but can not afford to tension, then all external force of bending moment for M point should be zero. That is,

\[ T'y - \frac{qx^2}{2} = 0 \]  

Where: 
q —— Uniformly distributed load on the sunken arch axis;  
T' —— Thrust of the sunken arch cross-section;  
x, y —— Respectively x, y coordinates of M point.

By static equilibrium equation, it is known that thrust of the sunken arch cross-section T' numerical equal with thrust T acting on the foot of the arch, but opposite direction, i.e., \( T = T' \) in numerical; And arch foot horizontal thrust T must meet the equation \( T \leq qDf/2 \), namely, in order to maintain the stability of the arch foot, the thrust T acting on the foot of the arch must be less than or equal to the maximum friction generated by the reaction force. Substituting equation (8) available the arch axis equation.

\[ y = \frac{x^2}{Df} \]  

Height at any point on the arch axis can be obtained from equation (9).

When \( x=D/2 \), the largest arch height \( y_{max} \) is:

\[ y_{max} = \frac{D}{4f} \]  

Where: \( y_{max} \) —— The maximum height of sunken arch;  
D —— The maximum span of sunken arch, as shown in figure 2. Can calculating by equation (6);  
f —— Sturdy coefficient of slippery bed, similar to the sturdy coefficient in Protodyakonov’s mountain rock pressure theory.

<table>
<thead>
<tr>
<th>Slippery bed category</th>
<th>Extrahard</th>
<th>Superhard</th>
<th>Hard</th>
<th>Harder</th>
<th>Ordinary</th>
<th>Softer</th>
<th>Soft</th>
<th>Loose</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sturdy coefficient f</td>
<td>20</td>
<td>15</td>
<td>8~10</td>
<td>5~6</td>
<td>3~4</td>
<td>1.5~2</td>
<td>0.8~1</td>
<td>&lt;1</td>
</tr>
</tbody>
</table>

5 Tunnel-landslide minimum safe distance

When the tunnel plastic zone is away from the landslide peristaltic squeeze region, which subject to a smaller influence of the landslide peristaltic squeeze, and the tunnel support design can according to the situation of the upper no landslide consideration; But when tunnel plastic zone in the landslide
peristaltic squeeze region, tunnel will be in the interaction of landslide peristaltic squeeze pressure, and we must consider the impact of the landslide peristaltic exert on the tunnel, that is, aside from ground pressure, the tunnel will also subject to landslide squeeze pressure. Therefore, the minimum safe distance \( H \) from tunnel vault to sliding surface must be greater than the sum of tunnel plastic zone and landslide maximum perturbation range. Namely:

\[
H > (R_{p_{\text{max}}} - r_0) + \frac{y_{\text{max}}}{\cos \beta}
\]  \hspace{1cm} (10)

By equation (5, 6, 9) and (10):

\[
H > r_0\left[e^{\left(\theta - \rho\right) \cot(45^\circ + \frac{\varphi'}{2})} - 1\right] + \frac{1}{4f \cos \beta} \left[ 2r_0 e^{\left(\theta - \rho\right) \cot(45^\circ + \frac{\varphi'}{2})} + Ah \right]
\]  \hspace{1cm} (11)

By equation (11), it is known that the safe distance \( H \) is related to tunnel radius \( r_0 \), slippery bed sturdy coefficient \( f \) and its internal friction angle \( \varphi \), slip mass internal friction angle \( \varphi' \) and its the thickness \( h \), and angle of the line connecting the two intersection point of fracture surface on the slip surface with the horizontal \( \beta \). In the case of the other parameters is constant, safe distance \( H \) with tunneling radius \( r_0 \), slip mass thickness \( h \) of a linear relationship. The nature of geo-material has an important effect on safe distance, consequently, it is necessary of a sensitivity analysis of safe distance \( H \) with internal friction angle and slippery bed sturdy coefficient \( f \), and the results as shown in Figure 4 ~ Figure 5.

Figure 4 shows that, whatever safe distance \( H \) change rate or sensitivity, angle of internal friction in slippery bed impacted than slip mass more sensitive. Sensitivity of safe distance \( H \) decrease linearly with the internal friction angle change rate increase, illustrate with the increase of internal friction angle impact on safe distance \( H \) decrease gradually. The initial sensitivity coefficient were 0.49 and 0.282, respectively, show that small changes of internal friction angle in the initial stage can cause safety distance has a great error, and the calculation must be based on the actual situation and choice a reasonable internal friction angle.

Figure 5 shows that, when \( f > 5 \), the sensitivity coefficient change rate is very small, all less than 1%. However, when \( f \) located in 0.8 and 5, the sensitivity coefficient change rate is relatively large, especially when \( f \) located in 0.8 and 3, safe distance change with the slippery bed sturdy coefficient is obvious. Illustrate the sensitive coefficient of safe distance with slippery bed sturdy coefficient is located in 0.8 and 5. Corresponds to the commonly used engineering rock classification method, the tunnel surrounding rock were divided into I ~ VI level. Table 2 shows the sturdy coefficient corresponding to different level of surrounding rock.
Fig. 4 Relationship of safe distance $H$ change rate and sensitivity with internal friction angle change rate

Note: $r_0=6.2m$, $h=30m$, $f=3$, $\beta=30^\circ$, $\phi_0=18^\circ$

![Graph of safe distance vs. slippery bed sturdy coefficient](image)

Fig. 5 Relationship of safe distance $H$ and sensitivity with slippery bed sturdy coefficient $f$

Note: $r_0=6.2m$, $h=30m$, $\phi=35^\circ$, $\beta=30^\circ$, $\phi'=30^\circ$, $f_0=0.8$

![Graph of sensitivity vs. slippery bed sturdy coefficient](image)

<table>
<thead>
<tr>
<th>Rock classification</th>
<th>I~II</th>
<th>III</th>
<th>IV</th>
<th>V</th>
<th>VI</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sturdy coefficient $f$</td>
<td>$&gt;8$</td>
<td>5~6</td>
<td>3~4</td>
<td>1.5~2</td>
<td>$&lt;1$</td>
</tr>
</tbody>
</table>

Table 2 sturdy coefficient of the surrounding rock

Table 2 shows that safe distance $H$ is insensitive to I~III class of surrounding rock, but very sensitive to IV ~ VI level. Surrounding rock is generally poor in landslide area and its sturdy coefficient located in sensitive interval. The calculation of security in the sensitive areas must choose a reasonable slippery bed sturdy coefficient in order to get satisfactory results.

Must be pointed out that the safety distance calculated by (11) was derived by the assumption that tunnel and landslide interaction and the safety distance is the perturbation range of them, not just the tunnel-landslide stress or displacement variation range. Obviously, tunnel-landslide stress or displacement variation range is much greater than the perturbation range, but we are more concern about the security of the tunnel. A large portion of stress adjustment or displacement change area located in elastic region, and the elastic region with a certain self-supporting ability and with little effect on tunnel safety. However, perturbation zone or plastic zone have lower stability, the region instability will have a direct impact on tunnel safety. If the distance of tunnel vault from sliding surface is less than the safe distance calculated by (11), engineering measures must be considered in tunnel construction process to protect the safety of the tunnel. For other cases, just put to use these construction methods that disturb the landslide small as far as possible in tunnel construction process and don't need to large-scale engineering measures can guarantee tunnel safe.

![Numerical calculation model of tunnel beneath the sliding surface](image)
In order to verify the correctness of the algorithm, numerical simulation analysis of tunnel beneath landslide were conducted. The numerical calculation model shown in Figure 6, in which the tunnel radius is 6.2m, landslide thickness is 60m, and the dip angle of main slide is 30°. Table 3 shows the numerical model parameters.

Table 3 numerical model parameters

<table>
<thead>
<tr>
<th>Geo-materials</th>
<th>bulk density/γ (kN/m³)</th>
<th>Elastic modulus / E(GPa)</th>
<th>Poisson's ratio /μ</th>
<th>internal friction angle /φ(°)</th>
<th>Cohesion /C (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slippery bed</td>
<td>20</td>
<td>2</td>
<td>0.28</td>
<td>40</td>
<td>200</td>
</tr>
<tr>
<td>Slip mass</td>
<td>19</td>
<td>0.8</td>
<td>0.30</td>
<td>35</td>
<td>100</td>
</tr>
<tr>
<td>Sliding Zone</td>
<td>18</td>
<td>0.2</td>
<td>0.32</td>
<td>30</td>
<td>10</td>
</tr>
</tbody>
</table>

According to geological conditions of the model, slippery bed sturdy coefficient f can take 3, substituted the corresponding parameters into equation (11) gives the safe distance H from tunnel vaults to the sliding surface is 12.8m, and H/D = 1.033 (D is tunnel diameter).

Figure 7(a) shows the plastic zone of tunnel vault is away from the sliding surface 1.5D, the lower side of the landslide plastic just located in the theoretical calculations position, that is, numerical analysis and theoretical calculation results basically consistent. According to the above theoretical calculations, geo-materials local in tunnel plastic zone straight up until the surface subject to the greatest impact, which have the trend of move to the tunnel free face direction. Displacement vector diagram in Figure 7(b) clearly show that this part of geo-materials displacement vector basically along with the vertical direction point to the tunnel free face, and formed a pseudo-broken surface about 45°-φ/2 angle on both sides of the earth column, outside of the pseudo-broken surface by less affected. This indicates that the basic assumptions in the theoretical derivation close to actual situation, the foregoing assumes is well-founded that the tunnel excavation changed the stress state within a certain range of the landslide, as long as the landslide stress state changed in the process of stress adjustment, we believe that the exist of tunnel have influence on the landslide.

Slope stress redistribution after the tunnel excavation, and the landslide began to move. Displacement vector of the zone most severely affected by the tunnel basically point along the vertical direction to the tunnel free face. Indicate that the assumption forces generated by landslide on tunnel is
mainly the vertical direction squeezing pressure, taking into account the system external forces to vertical direction and gradually passed to the tunnel is reasonable. Thus, the above calculation method of the tunnel-landslide system safe distance can provide a reasonable calculated value to ensure the safety of the system, provide a reference for the tunnel route selection and design.

6 Force analysis of tunnel support structure in tunnel-landslide system

For elastoplasticity surrounding rock, pressure on tunnel peripheral is the main reason of plastic zone formation and development. The greater ground pressure, the larger plastic zone in the surrounding rock. When plastic zone was formed, in order to maintain the balance of the plastic zone, need to provide some supporting force $P_i$, plastic zone will continue to develop until the supporting force $P_i$ can keep the tunnel stable. Assume excavation and set lining completed at the same instantaneous ($t = 0$). Due to the constant pressure $P$ acting on the surrounding rock appear a plastic zone with radius $R_p$. In order to calculate the tunnel supporting force in tunnel-landslide system, perturbation pressure $F_{\text{squeeze}}$ produced by landslide peristalsis must be calculated. Figure 8 shows the external force of tunnel-landslide system, in the diagram used the rigid limit equilibrium method of landslide analysis, number $i$ is the part of slide tunnel-landslide interaction.

In the diagram:
- $W_i$ — Weight of the $i$-th block of the slip mass;
- $Q_i$ — Force generated by the dynamic load acceleration in the $i$-th block;
- $E_i, E_{i-1}$ — Normal force act on the side face of $i$ and $i-1$;
- $X_i, X_{i-1}$ — Shear force act on the side face of $i$ and $i-1$;
- $\gamma$ — Angle of dynamic load acceleration vector with vertical direction;
- $F_i$ — Surface load act on the $i$-th block.

Fig. 8 External force diagram of tunnel-landslide system

After tunnel excavation, the landslide peristalsis will produce perturbation pressure on the tunnel, and the perturbation pressure is mainly on vertical direction. Therefore, calculation of the force primarily consider the vertical direction. The calculation model shown in Figure 8, in the model, disturbance region of tunnel and landslide were regard as a whole and its external forces are the landslide handed down which can be calculated in accordance with the limit equilibrium theory. According Sarma method [11] to calculate $E_i, E_{i-1}$ and $X_i, X_{i-1}$.

$$E_i = \zeta_{i-1} - M_{i-1}K_{i-1} + E_{i-1}e_{i-1}$$  \hspace{1cm} (12)

$$X_i = E_i \tan \phi_{S_i} + C_s d_i$$ \hspace{1cm} (13)

Where:
\[ e_{i-1} = \eta_{i-1} [\sec \phi_{3i-1} \cos(\phi_{Bi-1} - \alpha_{i-1} + \phi_{Si-1} - \delta_{i-1})] \]

\[ \zeta_{i-1} = \eta_{i-1} [(W_{i-1} + F_{i-1}) \sin(\phi_{Bi-1} - \alpha_{i-1}) + R_{i-1} \cos \phi_{Bi-1} + K_{i} \sin(\phi_{Bi-1} - \alpha_{i-1} - \delta_{i-1})] \]

\[ -S_{i-1} \sin(\phi_{Bi-1} - \alpha_{i-1} - \delta_{i-1})] \]

\[ \eta_{i-1} = \cos \phi_{Si} \sec(\phi_{Bi-1} - \alpha_{i-1} + \phi_{Si} - \delta_{i-1}) \]

\[ S_{i} = C_{S} d_{i} \]

\[ M_{i-1} = \eta_{i-1} W_{i-1} \sin(\gamma + \phi_{Bi-1} - \alpha_{i-1}) \]

\[ R_{i-1} = C_{Bi-1} b_{i-1} \sec \alpha_{i-1} \]

di——Length of the i-th sliding zone;
bi——Width of the i-th underside;
\( \alpha_{i} \)——Angle of the i-th underside with horizontal direction;
\( \delta_{i}, \delta_{i-1} \)——Angle of i, i-1 lateral sliding zone with vertical direction;
\( \phi_{Bi}, C_{Bi} \)——Strength parameters of sliding zone;
\( \phi_{Si}, C_{Si} \)——Strength parameters of lateral sliding zone;

External forces of tunnel-landslide system can be calculated according to (12), (13) and the landslide geometry conditions. Taking into account the squeeze pressure on tunnel in the process of landslide peristaltic subject to clamping action by both sides of the rock or soil and the tunnel is located beneath the landslide, disturbance squeeze pressure mainly in the vertical direction. Therefore, disturbance squeeze pressure on the tunnel is the resultant force of various external force of tunnel-landslide system in vertical direction, namely \( F_{\text{squeeze}} \).

\[ F_{\text{squeeze}} = W_{i} + F_{i} - X_{i} \cos \delta_{i} + X_{i-1} \cos \delta_{i-1} + E_{i} \sin \delta_{i} + E_{i-1} \sin \delta_{i-1} + Q_{i} \cos \gamma \] (14)

The perturbation pressure of tunnel-landslide system interaction can be obtained by equation (14). In tunnel support structure design, pressure on the lining is the sum of ground pressure and disturbance squeeze pressure, then the minimum support force must be provided to the tunnel is:

\[ P_{i} = K_{c} u_{0}^{n} = K_{c} (u_{p}^{0} - u_{0}) = K_{c} \left( \frac{M_{R}^{2}}{4G_{r}^{0}} - u_{0} \right) \] (15)

Where:
\[ K_c = \frac{2G \left( r_0^2 + r_1^2 \right)}{r_0\left[1 - 2\mu \right] r_0^2 + r_1^2} \]

\[ M = 2\left( P_{geo} + F_{Y_{squeeze}} / 2R_p \right) \sin \varphi + C \cos \varphi \]

Kc——Lining stiffness.

7 Conclusion

More and more attentions were paid to tunnel disease in landslide field by the engineering community, but there is no recognized solution in theory. In this paper, the author focuses on tunnel passing through beneath the landslide, and deduces the following understandings.

(1) Tunnel excavation in landslide field must be regarded tunnel and landslide as an overall interactive, that is, the tunnel-landslide system. It is constructed the concept of tunnel-landslide system which has an extremely important role in correct understanding the relationships of tunnel excavation and landslide peristaltic and analyze the deformation mechanism of them.

(2) Tunnel passing through beneath the landslide is a commonly used program to reduce the hazards of landslide and ensure the tunnel safety, therefore safe distance from tunnel vault to the sliding surface must be considered in the tunnel designing under the circumstances. Studies show that when the tunnel passes through beneath the landslide, force and deformation of the tunnel decrease with the increasing of distance from tunnel vault to the sliding surface. Safe distance is related to the tunnel radius, sturdy coefficient of slippery bed and its internal friction angle, slip mass internal friction angle and its thickness, and angle of the line connecting the two intersection point of fracture surface on the slip surface with the horizontal.

(3) Derived a formula of tunnel-landslide perturbation range, by the formula, safe distance (H) from tunnel vault to the sliding surface is computable, and verified by numerical simulation, shows that this method can obtain a satisfactory results.

(4) Analyzed the force of tunnel-landslide system, derived a formula of vertical perturbation pressure when tunnel-landslide interaction. And the pressure acting on tunnel support structure is the sum of normal state (without regard to the landslide ) ground pressure and tunnel-landslide vertical perturbation pressure.

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THE INTEGRATED CONTROL METHOD OF FOAM MIXED LIQUID INJECTION BASED ON EPB SHIELD FIELD PARAMETERS

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Based on experimental data on the EPB construction site, we study the relationship between the foam mixed liquid injection rate (m\(^3\)/h), the cutter torque fluctuation, the torque of the cutter, the cutter torque change rate over time and the tunnelling speed for the layer of one fixed soil body. We propose three new foam mixed liquid injection control methods including the method of cutter torque volatility, the method of foam injection volume-cutter torque fitting curve and the method of cutter head torque decrease rate. After discussing the three methods, we determine the integrated control method and carry out the on-site verification to make sure its effectiveness.

1 Introduction

In EPB foam method of soil improvement, the selection of rational foam mixed liquid injection can not only make full use of expensive vesicant, but also make the tunnelling speed increased. It’s of great significance for the shield construction. However, actually the phenomenon of shield-site foam injection too much or too little is inevitable. Especially for complex and volatile stratigraphy, the phenomenon is more likely to occur. The injection rate given by foam method of soil improvement mechanism and experienced shield machine operator often distorts due to the complexity of the EPB shield site conditions. We proceed to study accurate adjustment of the foam injection volume, deal with the collected EPB shield site data including on-site experimental data, and dig deeper into the relationship between the foam mixed liquid injection rate and the EPB shield tunnelling parameters. We begin in Section2 with discussing the limitations of the current foam injection control. In Section3 we
then describe relative related empirical research approach to allow us to provide stronger support for the conclusions. In Section 4 we analyze the experimental results and propose three new foam injection control methods including method of cutter torque volatility, method of foam injection volume-cutter torque fitting curve and method of cutter head torque decrease rate. In Section 5 we discuss in detail about the feasibility of the three methods. In Section 6 we combine the results of the discussions to determine the integrated control method and carry out the on-site verification. Finally, in Section 7 we affirm the important role of the integrated control method and some other significance of our research work. Then we make recommendations for ongoing and further work.

2 Current Practice and Research

Foam agent is a common soil improvement material. Various countries have been continuously developing foaming agent and optimizing the results of the foam soil body improvement. European countries, Japan and other developed countries study and develop the various properties of the foam agent, and study the performance of these blowing agents in laboratory and field. The amount of foaming agent in China is constantly growing. The current studies still tend to foaming mechanism and the development of new foam agents. The foam mixed liquid injection rate control method given by foaming mechanism and soil improvement effect at the present stage is difficult to achieve accurate control in engineering applications. We Select a soil improvement construction parameters by the experience of the technical staff and tests on unearthed samples in some EPB shield site. Tendency to the current domestic and international research leads to study on the foam mixed liquid injection rate accurate control less. Parts of the research results do not achieve the desired result.

3 Research Approach

In order to study the relationship between the foam mixed liquid injection rate (m³ / h) and other parameters, we do the following experiments. The radius of the Panel-Spoke cutter is 6.28m. We select a single soil layer with an obvious effect of foam soil body improvement. We change foam injection rate from insufficient to sufficient ones at different tunnelling speed, and observe the transformation of the cutter torque. In low speed, the records of torque sample can start with the zero foam injection. But in high speed, the records of torque sample must be from a certain rate. For some soil body, cutter torque tends to infinity over time when the injection rate is not enough. But in a certain period, the rising cutter torque is still within working torque range. In order to study the torque changes of this part, we adopt a new experimental method. First we let the injection rate reach enough. Then when the operating torque turns to minimum, change the injection rate. With the purpose of elimination of time lag impact, waiting for the torque to get a certain value we observe the rate of torque change in the case of different foam injection rate.

Fig1 is the foam mixed liquid injection rate-cutter torque fitting curve at different speed. The method of curve fitting is ‘Polynomial’.
4 Three New Methods of Controlling Foam Mixed Liquid Injection

4.1. Method of Cutter Torque fluctuation

When recording the torque sample point data, we find fluctuation range of different sample point data changes. Foam injection rate and tunnelling speed affects the torque fluctuation. In order to analyze this phenomenon, we process the data as follows. We connect the sample point data as record order, and shift image of the sample point data fluctuation to zero by the mean value of torque. Look at Fig.2, when the injection rate are respectively 0.4m³/h, 1.2m³/h and 1.5m³/h at speed of 15mm/min, the mean value of three sample points are respectively 2250KN*m, 1533KN*m and 1509KN*m. In the situation of V2, we do the same.

The analysis shows that, when the speed is fixed with a certain value, torque fluctuation range decreases with the increase of foam injection rate. But the use of excessive foam will lead to the increase of torque fluctuation range.

When observing the effect of driving speed, we process data in accordance with the above method. The analysis shows that, when the injection rate is fixed to a certain value, torque fluctuation range increases with the increase of driving speed.

Comprehensive analysis shows that torque fluctuation range increases with the increase of torque. The impact of foam injection rate is obvious. When the foam injection rate is enough and reasonable, the impact of driving speed will be not so obvious. Therefore, we get the first method, observing the minimal torque fluctuation range and the corresponding ideal injection rate.

4.2. Method of Foam Injection Volume-Cutter Torque Fitting Curve

The fitting curves are in Fig.1. For the convenience of description, the torque in a certain speed when the foam does not play a role is called Upper Limit Torque (short for \(T_P\)). We call the torque in a certain speed when the foam is enough for Lower Limit Torque (short for \(T_L\)), and the torque when it is declining for Transition Torque (short for \(T_T\)). In order to eliminate the impact of torque fluctuations, when deal with the horizontal segment corresponding to \(T_P\), We take the maximum fluctuation range value of \(T_P\), as the decline of \(T_P\). And use the corresponding injection rate to characterize the level segment. Similarly, we use the minimum fluctuation range value of \(T_L\) to process \(T_L\). 
Look at Tab.1, Max[|F1 - F2|, |F2 - F3|, |F3 - F1|]=70;
Max[FTPMax 1, FTPMax 2, FTPMax 3]=100;
Max[|F1 - F2|, |F2 - F3|, |F3 - F1|]<
Max[FTPMax 1, FTPMax 2, FTPMax 3].

Range of torque reduction is the same for the same soil body in our experiments. TP and TL increase with increasing speed.

The longer the horizontal section corresponding to the upper limit torque is, the more lag the starting point of the function of the foam injection rate is. The slope of the curve corresponding to the TP (K) is the function of cutter torque and foam mixed liquid injection rate. The absolute value of K decreases gradually with the increasing tunnelling speed. Similarly, we can say that, the longer the horizontal segment corresponding to TL is, the poorer the effect of the foam soil improvement is. But because of the limit of working torque, it is difficult to achieve the accurate control through studying the horizontal section of the curve to find the law of K. In order to achieve accurate control of the foam injection rate, we move the focus of the work to TP, finding the ending of the transition section curve (|K|=0). That is the starting point of the horizontal segment corresponding to TL. The fitting curve of those points at sufficient number of different driving speed is the theoretical curve of the accurate control. From the perspective of accurate control, we only care about that point and with the part of the curve close to it. So the incomplete curves at speed of V4 and V5 in Fig.1 are available.

4.3. Method of Cutterhead torque decrease rate

Because of working torque limit, the two incomplete curves in Fig.1 are incomplete. For some soil body, cutter torque tends to infinity over time when the injection rate is not enough. But in a certain period, the rising cutter torque is still within working torque range. Fig.3 is the curves indicate and Fig.4 is the experimental data curve using the new experimental method in Section 3.
In Fig. 1, the simple point corresponding to $T_T$ is the torque tending to a constant value when time tends to a certain value at certain foam mixed liquid injection rate. Only when the injection rate is greater than a certain value, and time tends to a certain value, the first derivative of the time on torque tends to zero. Then the torque tends to a certain value. The tunnelling torque is the higher order functions about time in our preliminary judgment. When the foam injection is not enough, the first derivative of the time on torque must not be zero. When the injection is in sufficient quantities, the first derivative may tend to zero. We use the implicit function to describe the relationship between foam injection volume, cutter torque and time.

$$f = u(T, I, t)$$

The boundary conditions of the ideal foam injection point are as follows:

$$\lim_{t \to t_0} \frac{dL}{dt} = 0$$

$$\lim_{t \to t_0} \frac{dT}{dt} = 0$$

$$\lim_{t \to t_0} \frac{d^2T}{dt^2} = 0$$

5 Development Practices Discussion about the above three methods

Through the first two methods, we can infer the foam mixed liquid injection volume-cutter torque curve is in the form of Fig. 5. In the segment of the curve between $(Q_1+Q_2-\Delta)$ and $(Q_1+Q_2)$ and in the rational segment between $(Q_1+Q_2)$ and $(Q_1+Q_2+\Delta)$, torque fluctuation changes little. So method of cutter torque volatility cannot realize the accurate control. When $\Delta$ gets a more large value, it will cause the waste of foam agent. When the rate is greater than $(Q_1+Q_2+\Delta_{\text{max}})$, the torque fluctuation range increases, entering the unreasonable range.

We can relatively accurately find the best foam injection point by using method of foam mixed liquid injection volume-cutter torque fitting curve. However, due to the fitting curve, every point of torque average processing

By using method of cutter head torque decrease rate, we can deal with the vertical curve segment. It plays an extremely important role in finding the starting point of the transition section. But based on
on-site control and experiment, it is difficult to characterize the higher derivative of the cutter torque-time function at the present stage. Method 3 is difficult to achieve in resent applications of foam injection control.

6 Determination of Integrated Control Method and Its Verification

In the above discussion, we initially affirmed the Method 2. In order to reduce the error of the ideal foam injection point we find, we use the Method1 to be supplemented. That is, when finding the ideal foam injection point, we take torque fluctuation into account to avoid too large deviation from the true point. There is another advantage of using this control method. Since the upper torque range is not the focus of the study, Method 1 to be auxiliary can help us avoid the experiments on the upper part of the curve in the normal tunnelling. By using the integrated control method, we get the ideal value of the foam mixed liquid injection rate at different tunnelling speed and the corresponding minimum torque as Fig.6 and Fig.7 show. After comparing the measured torque of the cutter and testing the unearthed, we confirm that application of this control method in the on-site construction can meet the requirements of soil improvement. Compared with the control method in traditional construction, the effect is remarkable to save the foam injection volume of 10%-25% approximately during driving speed from 15mm/min to 25mm/min.

7 Conclusions and Future Work

Different from the tendency of the soil improvement research at the present stage, we focus our research on the regulation of the EPB-site construction foam mixed liquid injection rate in this paper. Because the integrated control method of cutter torque volatility and foam injection volume-cutter torque fitting curve can well adapt to the control of on-site construction foam mixed liquid injection rate, the amount of vesicant drop and driving speed is improved. Indeed, the highlight of this work is not only the establishment of the integrated control method, but also the establishment of the soil evaluation standards specific to EPB-site construction after a large number of similar works. Considerable work still remains to be carried out in this area, including laws of the method of cutter head torque decrease rate and the applicability of the Control method for the composite soil layer.

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References

EXPERIMENTAL STUDY OF IN SITU RIGID BEARING PLATE TEST FOR GREY-GREEN SHALE

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Abstract: The grey-green shale is soft or very soft rock. In situ roundness rigid bearing plate creep was conducted in the experimental tunnel in order to get the rheological properties of the grey-green shale when it is used as foundations. Using the latest developed LSB-1A electro-hydraulic servo rheological test instrument achieved the rheological full test process control. The analyses of experiment data draw the general creep law and gave the failure mechanism of the whole process. The displacement calculation formula under rigid bearing plate was obtained as well as deformation modulus through back analysis based on the elastic J.Boussinesq problem. Burgers model is adopted to describe the grey-green shale’s rheological properties and viscosity parameters were gotten by optimizing inversion method.

Key words: rock mechanics; rigid bearing plate; in situ creep test; creep characteristics; parameter identification

1 Introduction

In situ bearing plate rheological compression test is the most commonly used among in situ rheological test methods. The bearing plate can be divided into rigid bearing plate and the flexible bearing plate depending on the nature of itself. In recent years, with China's large-scale water conservancy and hydropower project construction, mine development and implementation of western development strategy, many domestic scholars use in situ bearing plate compression rheological tests to study the rheological properties of the rocks. In order to get the compression creep parameters of the deep rock, YANG Wendong, ZHANG Qiangyong conducted in situ rigid bearing plate center hole deformation test, and deduced the generalized Kelvin model’s instantaneous elastic deformation of deep rock under circular rigid bearing plate which bears the uniformly distributed load, also obtained the creep parameters by the optimization inversion algorithm; LIU Yunfang, ZHOU Mi analyzed the bearing plate test methods, provided the displacement analytical expression for the rigid and flexible bearing plate inside and outside the measuring point from the perspective of elasticity; FEI Dajun, XIE Ruping and others did experimental research of the soft rock of a dam by rigid bearing plate center hole deformation test, analyzed the soft rock mass deformation with depth and pressure variation; ZHANG Qiangyong, WANG Jianhong etc. deduced compressive deformation formula of deep rock mass according to the field deformation test of large-scale rigid pressure-bearing plate of Dagangsshan dam zone, and by analyzing the deformation curve of loading and unloading, elastic modulus, deformation modulus and equivalent deformation modulus of test rock mass were calculated.

Gray-green shale in engineering is often treated as soft rock or very soft rock, easy weathering and easily changes to clay in presence of water, which rheological property is also very dramatical. Previous in situ rheological test few care for the soft and broken rock, but focuses on how to obtain the rheological parameters, rather than study the rheological parameters change over time and the mechanism. In this paper, an in situ rigid bearing plate compression rheological test was conducted. The process of the test was described in detail, and based on test results, the rheological character of weak and fractured rock mass was analyzed and its parameters was obtained through optimized inversion method.

2 Engineering background

The hydroelectric station is located in central Guizhou Province. The capacity of station power installed is 3000MW. Its navigation structure is located in the left bank of the Wujiang River and is one of the Project's three main buildings, upstream and downstream water level is 637m, 510m, and ship
The lift lifting height is 127m.

The foundation of navigation structures primarily located in the Ordovician group under paragraph (O1m1) micro-bedrock. The formation is grey-green shale with a small amount of thin layer of calcium powder fine-grained sandstone and thin bioclastic limestone, belong to soft rock or very soft rock, and deformation modulus is very low. The general trend of the rock is from 20° to 40° inclined to NW, dip about 45°. The rock stratum is strongly squeezed by tectonic compression. There are crumpled or small fold developed in local rock, and rock attitude changes dramatically. There are layers of dislocation or fracture zone in part of the rock foundation, and the traits are poor.

3 In situ rigid bearing plate compression rheological test

3.1 Choice of test site and excavation

The rheological test was conducted in the test tunnel. Thin to thick layered grey-green shale was select as test point. As the rock is broken and weak, loose surface layers have to be removed before test. To avoid disturbance of the original rock, the excavation was operated by hand. The final excavation surface should be no weathering, crumbling, and made sure the deviation of the plane between the highest and lowest points should be no more than 0.1d (d is the diameter of the plate). After the surface been prepared, scrub and rinse it with clean water to remove any loose particles or dirt caused by the smoothing operation. Construct the bearing pad using cement paste, and the maximum thickness is no more than 0.15 times the diameter of the pressure plates.

3.2 Test equipment installation

Test equipment installation shows in figure 1. We place rigid bearing pad, steel plate, jack, the pillar and steel plate above the bearing pad, and fill in reinforced concrete between the upper steel plate and the tunnel roof. Then exerting contact pressure to make the system contact closer.

LSB-1A electro-hydraulic servo rheological test instrument was specially developed for the test, which is composed by hydraulic jacks, hydraulic servo oil source, deformation measurement systems, load control systems, industrial computers, UPS power etc. And by using computer, it can realize the automatic control of the test process, including data acquisition and storage, loading apply and maintenance, etc.

Four dial indicators were placed uniformly on the bearing plate, and take its average results as rheological displacement.

3.2 Test methods and procedure

Stage loading method was adopted. Load level and stage loading scheme were determined according to the deformation modulus and destruction pressure that obtained by formerly rock deformation test and the rock load test. Electro-hydraulic servo load regulator system can effectively compensate the dropping pressure caused by fast deformation in the early period.

Take longitudinal deformation rate as a stable standard. The deformation is considered to be steady.
when under a certain level load, the average results of the four displacement sensor i.e. the deformation rate is less than 0.01 mm/h. Then apply next level load.

Four groups of compression rheological test were conducted on site, the first group failed due to the change of geological condition. We unload two load levels to 0 after stability, and observe its performance after unloading.

4 Test results and analysis

![Fig 2 LB-2 test point rheological curve](image)

![Fig 3 LB-3 test point rheological curve](image)

![Fig 4 LB-2 test point step load creep curve](image)

![Fig 5 the percentage of creep displacement account for total displacement of LB-2 under each load](image)

Figure 2 and Figure 3 show the whole process displacement-time curve of test points LB-2 and LB-3. Figure 4 shows the classification creep curves of point LB-2. Unloading to zero at level 0.6MPa and 1.5MPa of point LB-2, and apply to the next load level after stability. The following rules can be obtained by analysing the deformation-time curve:

1. At early loading period, instantaneous deformation can be seen, and the higher the load level, the greater the instantaneous displacement. As time goes by, creep deformation develops dramatically.

2. At low stress level, creep soon goes steady. There are only stage I and stage II, and the creep rate of stage II after steady is close to zero. At higher stress levels, the creep rate of the stage II increases with the increasing stress level. When reaches a certain level, the displacement grows rapidly, and the curve goes all the way up, that lead to the broken of rock. There is a stress threshold, when the stress is greater than the threshold σs, creep stage I, II, III can be observed, and the rock lose stability quickly. This stress threshold can be equivalent to a certain long-term strength of the rock at some extent. And according to the curve of LB-2 and LB-3, we can speculate that the stress threshold is about 2.3MPa.

3. Unloading to 0 at load level 0.6MPa and 1.5MPa of point LB-2 after stability, a sudden elastic recovery can be seen, and the recovery increases with time until next stability. There is permanent deformation after stability. At low stress level, the creep rate soon decreases to 0 and after unloading, the recovery of the deformation is almost equivalent to the creep deformation. While at high stress level, final permanent deformation is greater than the initial instantaneous deformation at this level, i.e. the creep deformation cannot be fully recovered. The causes of the phenomenon are that rocks are much fractured at test point. Rocks are pushed close at low stress level. Deformation is mainly
composed by the diastrophism of the structure planes or fillings, creep deformation can be easily recovered when unload. While at high stress level, deformation also includes rock deformation. With the development of deformation, micro-cracks are generated within the rock and unrecoverable deformation gradually increases.

(4) Figure 5 shows the relationship of the percentage that creep displacement accounts for the sum of instantaneous elastic displacement and creep displacement with load level. With increased stress level, creep displacement becomes the dominant gradually. The rock will collapse when the proportion of creep displacement reaches to about 90%.

5 Identification of rheological model and parameter inversion

5.1. Inversion of elastic parameters

The theoretical basis of the bearing plate method is a uniform, continuous, isotropic semi-infinite elastic Boussinesq problem under local surface force. The displacement calculation formula can be obtained by the integral of concentrated force P points within the area of the rigid bearing plate. By reference\(^2\), the calculation formula of surface vertical displacement of solid round bearing plate is:

\[
\delta_v = \frac{q_0 R^2 \pi (1 - \mu^2)}{2 E_0} 
\]

(1)

Where \(q_0\) is the stress level of bearing plate, \(R_0\) is the radius of bearing plate, \(E_0\) is rock deformation modulus. Thus:

\[
E_0 = \frac{q_0 R^2 \pi (1 - \mu^2)}{2 \delta_v} 
\]

(2)

Table 1 Elastic parameter inversion results of LB-2

<table>
<thead>
<tr>
<th>Load level (MPa)</th>
<th>Displacement (mm)</th>
<th>Deformation modulus E0 (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.6</td>
<td>3.318</td>
<td>64.38</td>
</tr>
<tr>
<td>1.2</td>
<td>8.547</td>
<td>49.98</td>
</tr>
<tr>
<td>1.5</td>
<td>11.000</td>
<td>48.55</td>
</tr>
<tr>
<td>1.7</td>
<td>13.898</td>
<td>43.55</td>
</tr>
<tr>
<td>2.0</td>
<td>16.025</td>
<td>44.43</td>
</tr>
<tr>
<td>2.3</td>
<td>18.399</td>
<td>42.57</td>
</tr>
<tr>
<td>2.6</td>
<td>20.950</td>
<td>42.48</td>
</tr>
<tr>
<td>2.9</td>
<td>24.922</td>
<td>41.43</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>47.17</strong></td>
<td></td>
</tr>
</tbody>
</table>

According to equation (2), rock deformation can be calculated by instantaneous settlement and each load level. Take point LB-2 as an example, the elastic parameters under different load levels of different test points are given in Table 1.

Form the rock deformation modulus of Table 1, we can see that \(E_0\) shows a decreasing trend with the increase in stress level, which indicates that the rocks begin to weak gradually at the beginning of loading. This is different from our common sense that at initial loading stage, rock deformation modulus increases because of rock was pushed closer. This also shows that rocks here are extremely incompact. The rock mass break when its deformation modulus decreases gradually to the point that it is not strong enough to resist the load increasing rate.

4.2. Inversion of viscosity parameters

First we have to determine the rheological model describing the rock mass before the inversion of viscosity parameters. We can see that the rocks show a significant viscoelastic property according to rock rheological theory and test curves. Burgers model is composed by Maxwell model and Kelvin model in series, shows a property of instantaneous elastic strain and viscous flow performance when loaded and instantaneous elastic recovery and elastic aftereffect
when unloaded. Therefore, it can be used to describe the rheological properties of grey-green shale at test point. Its Constitutive equation is:

\[ \dot{\varepsilon}(t) = \frac{\sigma_y}{E_0} + \frac{\sigma_y}{\eta_k} t + \frac{\sigma_y}{\eta_m} [1 - \exp\left(-\frac{E_0}{\eta_k} t \right)] \]  

(3)

Under the action of the circular rigid bearing plate, we consider the vertical displacement under bearing plate within the surface of rock mass is equal. Thus derive the viscoelastic analytical solution of rock mass displacement under rigid bearing plate of Burgers model[11]:

\[ w(t) = \frac{\pi pR^2}{8} \left( \frac{1}{K} + \frac{t}{\eta_k} + \frac{1}{G_0} \left( 1 - e^{-\frac{t}{\eta_k}} \right) \right) \left( \frac{p_2}{a_2} - \frac{a_1}{a_2} \right) e^{-\frac{a_1}{a_2} t} - \frac{q_1 (\sigma_0 e^{-\frac{a_1}{a_2} t} - \sigma_1 e^{-\frac{a_1}{a_2} t})}{(3Kp_2 + a_2) (a_2 - a_1) K} \]  

(4)

where, \( a_1 = \frac{b_1 + b_2}{2b_2}, a_2 = \frac{b_1 - b_2}{2b_2}, b_1 = 3Kp_1 + q_1, b_2 = 3Kp_2 + q_2, b_3 = \sqrt{b_1^2 - 12Kb_2}. \)

\[ p_1 = \frac{\eta_k + \eta_m}{G_0}, p_2 = \frac{\eta_k \eta_m}{G_0}, q_1 = \eta_k, q_2 = \frac{\eta_k \eta_m}{G_0} \]

Bulk modulus \( K \) and shear modulus \( G_0 \) can be derived from \( E_0 \):

\[ K = \frac{E_0}{2(1 + \mu)} \quad \sigma_k = \frac{E_0}{3(1 - 2\mu)} \]

By the above formula (4), we still have three unknowns \( G_1, \eta_m, \eta_k \) if we want to get the analytical solution of displacement. And this can be obtained by optimization inversion method. If we take design variables \( X = (G_1, \eta_m, \eta_k) \), the objective function can be wrote as follows:

\[ f(X) = \sum_{i=1}^{n} [w(t)_c - w(t)_i]^2 \]  

(5)

where, \( n \) is the number of measured displacements; \( w(t)_i \) is the displacement that calculate by formula (4); \( w(t)_c \) is the measured displacement at time \( t \). To simplify the problem, we can give the estimated upper and lower limit of variables according to elastic inversion results and engineering analogy design, and establish the constraint conditions:

\[ a_i \leq X_i \leq b_i \quad (i=1,2,3) \]  

(6)

where, \( X_i \) is the \( i \)-th design variable, \( a_i, b_i \) are the upper and lower limits of design variables.

Using the complex method, we can program the above formula into the program, and work out the optimizing algorithm. Take point LB-2 as an example, we get the inversion results of viscous parameters in table 2.

<table>
<thead>
<tr>
<th>Stress level(MPa)</th>
<th>G1(MP)</th>
<th>ηK(MPa)</th>
<th>ηM(MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.60</td>
<td>238.58</td>
<td>451.89</td>
<td>6297.30</td>
</tr>
<tr>
<td>1.20</td>
<td>76.98</td>
<td>70.80</td>
<td>7591.90</td>
</tr>
<tr>
<td>1.50</td>
<td>104.50</td>
<td>92.10</td>
<td>7577.50</td>
</tr>
<tr>
<td>1.70</td>
<td>170.62</td>
<td>114.60</td>
<td>12552.70</td>
</tr>
<tr>
<td>2.00</td>
<td>161.05</td>
<td>344.44</td>
<td>9484.03</td>
</tr>
<tr>
<td>2.30</td>
<td>196.33</td>
<td>617.30</td>
<td>9278.50</td>
</tr>
</tbody>
</table>

| 平均值 | 158.01 | 281.86 | 8796.99 |
6 Conclusion

This paper gives a detailed introduction to the procedures of roundness rigid bearing plate compression rheological test. And by analyse the test results, we draw some laws of the rheological properties of grey-green shale as follows:

1. Grey-green shale shows a significant rheological properties and obvious viscoelastic properties.
2. The percentage of creep displacements account for the sum of displacements increases with the increase of stress level. The rock mass breaks when creep displacement becomes dominant.
3. The grey-green shale creep rate decrease to zero quickly at low stress level, the higher the stress level, the higher the stable creep rate. And there is a stress threshold that when it is exceeded, thus arises accelerate creep phase, and the rock mass breaks as a result.
4. The grey-green shale is soft rock or extremely soft rock, its deformation modulus decreases with the increase of stress level.

References
STUDYING THE TWO FIELDS COUPLING MECHANISM AND ITS TREATING MEANS OF TUNNEL UNSTABILITY OF COASTAL CITY

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The coastal city tunnel construction often meets with the seepage and water intrusion problems. The paper took the tunnel of water rich zone of Dalian city in China as engineering background, carried out the laboratory and numerical simulation. Test and simulation results stated that the fluid-solid interaction made the surrounding rock more unstable, especially, because of the tunnel face being not supported, the dynamic water decreased the safety coefficient obviously. Stress field and hydraulic field have complex interaction, one of them is aggravating will makes the other also has unbalance trend. Therefore, the fluid-solid interaction should be considered in construction of tunnel in rich water area. According to the engineering condition, the construction treating means such as strata injection, dewatering, interception of water etc are suggested. The multivariate information monitoring should be adopted and feedback analysis can avoid some water intrusion accident on a great degree.

1 Introduction

For tunnels in rich water area, groundwater is the main factor affecting the stability of surrounding rock. In the tunnel excavation process, the complex coupling action of stress-seepage aggravates deform and instability of surround rock. A. Biot began the research of fluid-solid coupling of rock and soil from 1941, and he proposed the relation between seepage field and effective stress field [1]. R. W. Lewis proposed full coupling model based on Biot consolidation theory and adopted uniform physical parameters in his analysis [2, 3]. Li Tingchun simulated a subsea tunnel based on three dimensional Biot consolidation theory, and the results shown that fluid-solid coupling affects the displacement and plastic zone of surrounding rock evidently [4]. Dalian city subway of China was constructed from 2010, the location is shown in Figure 1. Many bid sections have complex geological condition, the ground water problems are serious (Figure2). The paper will study the flow field and interaction mechanism of seepage-stress in the region, and suggested the proper treating construction meanings.

The Xuehai interval tunnel of bid section110 beneath the Lingshui River near the Maritime University, Lingshui River origins from the west mountain, flows across the city from the southeast to the sea. Small reservoirs were built upstream, the capacity of them is 1,167,000m³, and annual water supply capacity is about 470,000m³ 3. The width of Lingshui River is about 50m, the river is an intermittent river, downstream normal is dry, and a drainage channel downstream of discharge volume soared during the rainy season. During the investigation the water depth was about 1.0m. Case of the downstream estuary has salt water intrusion at high tide. The main types of groundwater are Quaternary bedrock fissure water and pore water, mainly occur in the Quaternary strata of the pores and cracks in the bedrock. For differences in the formation permeability, gravel layer and the bedrock of the water slightly with pressure. For the rock fissures, pore water and fracture water localized with connectivity.
Groundwater has no corrosion to the concrete structure has a weak resistance to the steel reinforced concrete structure, has a moderately corrosive to the steel structure.

2 Fluid solid coupling test

Firstly, we adopted typical rock specimen and carried out fluid solid coupling test. Use the advanced tri-axial seepage-stress coupling experiment instrument to study the relation between coefficient of permeability and stress. The instrument (Figure 3) includes three independent apparatuses, and its characteristics are as follows: 1) The water seepage system can provide the maximum hydraulic pressure of 70MPa and keep the steady-state seepage; 2) Confining pressure and pore water pressure are separated by rubber sleeve. Based on the laboratory, we can get the equation which reflects the relation between coefficient of permeability and stress. The equation can be combined with the Biot consolidation numerical method. The specific process has been often expounded before [5, 6, and 7]. The typical curves reflecting seepages and confining pressure are shown in Figure 4.

Based on the laboratory we can get the equation reflecting relation between coefficient of permeability and stress. The equation can be combined with the Biot consolidation numerical method. The specific process has been often expounded before. The result of test can get the parameters of the calculation control equations, based on which the numerical simulation method is gotten. The seepage stress coupling simulation result for tunnel face is in composition.

Figure 1 The subways in China

Figure 2 The water inrush of tunnel face

Figure 3 The testing machine and specimen

Figure 4 The seepages with different confining pressure
3 The field flow field and tunnel face fluid-solid action

3.1 Region flow field simulation by Visual MODFLOW

The watering in crosswise passage of landslide region is pumped in vertical shaft, which is numerically simulated. Visual MODFLOW is application software that able to create three-dimensional groundwater flow and contaminant models, with a friendly user interface. For the engineering geological characteristics and morphology, the profiles calculated in the region include: Artificial Quaternary Holocene collegial (Q₄ml), Quaternary Holocene alluvial layer (Q₄al + pl), Quaternary Pleistocene alluvial layer (Q₃al + pl), to update the Quaternary President slope diluvium (Q₃dl + pl), the geological conceptual model section is shown in Figure 5. External hydraulic head, soil permeability, water rate and other parameters have a great impact on the groundwater flow. Except the Vertical permeability Kz, other coefficients can be measured by simple experimental means. For the determination of Kz, We assume that each unit of horizontal and vertical anisotropy ratio (Kx / Kz) is 10 to calculate the value of each unit Kz.

![Figure 5 Model of the generalization of geological profile](image)

As the time between collapse and pump are too long, we irrigation the tunnel full before the simulation to save time. Simulation of pumping process in strict accordance with the ahead 38 hours of the pumping processes, and then stops the wells, 32 hours of ground water flow simulation was observed. The simulation time for a total is 100 hours. As it is shown in Table 1, positive means Irrigation, negative means pump.

<table>
<thead>
<tr>
<th>Start time (hour)</th>
<th>End time (hour)</th>
<th>Pump rate (m³/h)</th>
<th>Start time (hour)</th>
<th>End time (hour)</th>
<th>Pump rate (m³/h)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>30</td>
<td>0</td>
<td>36.4</td>
<td>37.8</td>
<td>-32</td>
</tr>
<tr>
<td>30</td>
<td>31.7</td>
<td>-160</td>
<td>37.8</td>
<td>43.5</td>
<td>-52</td>
</tr>
<tr>
<td>31.7</td>
<td>32.2</td>
<td>-80</td>
<td>43.5</td>
<td>51</td>
<td>-31</td>
</tr>
<tr>
<td>32.2</td>
<td>32.4</td>
<td>-160</td>
<td>51</td>
<td>55.2</td>
<td>-52</td>
</tr>
<tr>
<td>32.4</td>
<td>35.7</td>
<td>0</td>
<td>55.2</td>
<td>60.2</td>
<td>-40</td>
</tr>
<tr>
<td>35.7</td>
<td>36.4</td>
<td>-20</td>
<td>60.2</td>
<td>100</td>
<td>0</td>
</tr>
</tbody>
</table>

Figure 6 shows the hydraulic head in the pumping process, hydraulic head assumes descending from Lingshui River to the left side and the rate of it is slow. It can be seen that there is a negative zone on the right cutting face in the pimping process. The hydraulic head decrease fast in this zone. On the left side, a partial hydraulic head also increases. Because the tunnel makes the region become a large connected device in a short time, pass the head of Ling River to the left side, and makes their heads the same. The groundwater will flow shrewd in this process, and make the terrestrial stress heightened in
some places for a short time. When the flow is steady again, the hydraulic head will become normal. But there is a circulation of loading and unloading for the soil mass in the process. The soil mass is disordered and swept. Add discomposure factors to the accident. It is very indispensability to make injection in this accident.

Figure 7 shows the comparison between the hydraulic head values of the site and simulated hydraulic head values. Both the trends and the value range can be found roughly. It means this model can well simulate the situation on the spot. The main recharge is from the Ling River through the gravel layer. The original assumption that the recharge is from the seawater intrusion cannot be established. The recharge rate is approximately 19m\(^3\)/h. The simulation technicians can reasonably establish in precipitation scheme accordance to the model or find the flooding point.

![Figure 6 hydraulic head in the pumping process](image1)

![Figure 7 Comparison between heads](image2)

### 3.2 Tunnel face collapse mechanism

Based on test results, we can construct seepage stress coupling numerical model. The basic unstable seepage flow difference equations considering compressible water and rock soil matrix are as follows.

\[
\frac{\partial}{\partial x} (k_x \frac{\partial h}{\partial x}) + \frac{\partial}{\partial y} (k_y \frac{\partial h}{\partial y}) + \frac{\partial}{\partial z} (k_z \frac{\partial h}{\partial z}) = p g (\alpha + n \beta) \frac{\partial h}{\partial t} = S_s \frac{\partial h}{\partial t} \quad (1)
\]

In the above formula, \(h\) is hydraulic head function, \(x, z\) are space coordination, and it is time coordination. \(k_x, k_z\) are coefficients of permeability respectively along \(x\) and \(z\) directions. \(S_s\) is the unit water-storage capacity. For an element of surrounding rock, each stress component should meet the condition of static balance. The effective stress balance equation in three dimensional cartesian coordinate system is:

\[
\frac{\partial (\sigma_{ij} + \delta_{ij} \rho p)}{\partial x_i} + f_j = 0 (i, j = 1, 2, 3) \quad (2)
\]

Where \(\sigma_{ij}\) is the effective stress tensor, \(\sigma_{ij} + \delta_{ij} \rho p\), \(\sigma_{ij} + \delta_{ij} \rho p\) is the total stress tensor. \(p\) is pore water pressure. \(0 \leq a \leq 1\) is equivalent pore water pressure coefficient. \(\delta_{ij}\) is the symbol of Kroneker. \(f_j\) is volume force.

Hydraulic field unbalance and dynamic water pressure is important factors of inducing tunnel face disability. We can simplify the tunnel face to 2D problem as Figure 8. The soil body under saturation line is acted by force D, which can be expressed by the multiplication of seepage force and seepage area.
Figure 8. Tunnel face disability slice method

Figure 9. Tunnel face disability simulation

\[ D = \gamma_w \cdot i_A \cdot F \]  

(3)

Where, \( \gamma_w \) is the water weight density, \( F \) is the area of the soil body under saturation line. Dynamic water action force \( D \) can be considered acting at the centroid of area \( F \), direction is parallel to line AC, while the force produced slip moment, and the stability factor according to slice method is as follows:

\[ k = \frac{\sum_i (w_i \cdot \cos \alpha_i \cdot \tan \phi + c) + D_r \cdot R}{\sum_i w_i \cdot \sin \alpha_i + D_r \cdot R} \]  

(4)

Where \( r \) is the arm of force \( D \) to the center of slip circle, \( w_i \) is weight of soil slip, \( \alpha_i \) is the included angle of the tangent line of the circular arc where slip is and the horizontal line. It is shown that, the higher the underground water is, the dynamic water force is more serious, which obviously decrease the slip body safety factor.

4 Treatment method and construction means of crossing river section

4.1 Strata injection

After the local collapse of Dalian subway 100 bid sections, quickly adopted two rows of steel piles plugging tunnel face, then back filled sand stones. According to landslide body and pipe lines display, adopts the sleeve stopper pipe to make injection, in order to insulate the supply environmental water. Besides the landside region reinforcement, in the advanced region of crossing river, also adopts injection scheme, arrange the injection points with the distance of 1.5m. The landside region reinforcement is shown in Figure 10, part of the cross river region reinforcement is shown in figure 11.
4.2 Dewatering and interception of water

According to the information leading integrative geology prediction, combining excavated section size, technology capability and economic feasibility, synthetically considering tunnel and construction measures. Because groundwater riverbed, two sides of the river should be set dewatering pits in order to remove crevice water and decrease the influence to the construction. In the upstream and downstream scopes which are influencing the tunnels, the concrete cut-off walls are set. The wall top should be 0.8 meters higher than stream bed, 3 steel pipes with 44 meters and 200 cm diameter are used to drainage, making the open water flow off span the tunnels interval region through concrete pipes.

4.3 Feedback analysis based on monitoring data

The occurring of surrounding rock collapse and water intrusion are corresponding to the destroy process of underground engineering, if we regard the surrounding rock collapse and water intrusion as dynamic system, the monitoring data of surrounding rock displacement and seepage flow include the inner developing regulation. Fully taking usages of the monitoring data and feedback analysing and adjusting the construction schemes, which is a feasible mean to prevent the disasters. By collecting the information of the surrounding rock, evaluate the stability of tunnel surrounding rock, so as to appraise the rationality of the construction scheme. Measurement items include the required and the optional items. Compared with critical values, the changing ratio of stress and displacement have more definite physical significance, it also can reflect the hydraulic effect and creep effect. If displacement and stress changing velocity v equates zero, the subway tunnel tended to be stable, on the contrary, if v equates a constant C or increasing persistently, subway tunnel tended to collapse.

5 Conclusions

Aiming at the seepage and water intrusion problems of Dalian subway 100 bid sections, the tests and numerical simulations are carried. The flow and recharge of groundwater have great influence on the safety and construction. Due to the complexity of underground geological conditions, and the restrictive of the strata conditions, the calculation is difficult. In this paper, use model to simulate the geological conditions, adjust the parameters according to the differences between Calculated head values and field monitoring data. Makes the last two values in the range of trends and roughly the same to get an accurate numerical model simulated field conditions.

The fluid-solid interaction makes the surrounding rock more unstable. The paper analysed the seepage-stress coupling mechanism of the disability process of the tunnel face, mechanics field and hydraulic field acted reciprocally, one is losing balance, and the other also is trending to disability. The dynamic water action will aggravate the destabilization of the surrounding rock. Therefore the fluid-solid interaction should be considered in rich water area tunnel analysis.

For collapse treating and prevention, proposed the principle of keeping hydraulic pressure balance and avoiding disturbing tunnel face. According to the engineering actuality of the section of crossing river, suggested the means such as strata injection, dewatering and interception of water. Especially, the monitoring of surrounding rock displacement and seepage flow should be concerned, so as to feedback adjust the construction scheme in time.

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155–164.


SATURATED/UNSATURATED, UNSTEADY SEEPAGE ANALYSIS OF ROCK THREE DIMENSIONAL FRACTURE NETWORK

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Unsaturated seepage of fractured rock is common and objective in natural, the research on unsaturated seepage of fractured rock by discrete fracture network model could reflect truly the actual flow state of fracture network. Currently two key issues are included in the research on unsaturated seepage of fractured rock, namely, the generation of seepage network and mesh generation of seepage network. To these problems, three-dimensional fractured rock flow network search algorithm was formed by improving the existing three-dimensional block cutting algorithm and increasing the judgment of waterproof properties of fracture, etc. The problem of mesh generation caused by irregular fracture was solved by constrained Delaunay triangulation technique that seemed these fractures as constrained conditions. Then based on relationship function between unsaturated seepage parameter and unsaturated seepage by Prof. Zhang, the finite element equation was established by Galerkin method on unsaturated/unsaturated unsteady seepage analysis of fractured rock network. It was proved that the direction of water flow in fracture network was in accordance with realistic flow direction, and the calculated values of measuring point head water was in well keeping with measured values in steady state, the maximum error of less than 1.07%, which showed that the method could produce objective and accurate percolation network, but also could generate a reasonable computational mesh; and the calculations reflecting the changes of water flow could accurately reproduce the actual fracture network trends in water flow and movement rules.

1 Introduction

In recent years, with the development of porous medium unsaturated seepage and fractured rock saturated seepage, it is becoming clear that rock slope landslide in the rains, the landslide of underground tunnel and gallery, rock slope failure which be caused by flood discharging atomization, etc, are closely related to the unsaturated seepage in the fractured rock mass[1]. Therefore, it was tested and theories studied by many scholars at home and broad in the mid 1980s, the main topics are as follows: (a) the test and theory research on single fracture unsaturated seepage, the main focus on the measurement and determination of its hydraulic parameters; (b) the various mathematic model to solve unsaturated seepage of fractured rock mass and the corresponding numerical analysis.

At present, mathematical models of unsaturated seepage in fractured rock can be classified into [2]: continuum seepage model, discrete fracture network seepage model and continuum-discrete coupling seepage model. Among them, the research on unsaturated seepage of fractured rock by discrete fracture network model could reflect truly the actual flow state of fracture network. There are two key problems on the unsaturated seepage research of fractured rock mass by discrete fracture network seepage model [3], one is how to build the seepage channel by fracture network, and another is how to solve the seepage problem of any profile region on two dimensional flow of fracture surface.
To these problems, three-dimensional fractured rock flow network search algorithm was formed by improving the existing three-dimensional block cutting algorithm and increasing the judgment of waterproof properties of fracture, etc. The problem of mesh generation caused by irregular fracture was solved by constrained Delaunay triangulation technique that seemed these fractures as constrained conditions. Then based on relationship function between unsaturated seepage parameter and unsaturated seepage by Prof. Zhang [4], the finite element equation was established by Galerkin method on unsaturated/unsaturated unsteady seepage analysis of fractured rock network. Finally, the correctness of unsteady seepage analysis in fractured rock mass is verified through the test data by Grenoble.B.A[5] in 1989.

2 The building of three dimensional seepage network and its mesh generation

2.1 The building of seepage network

Rock mass seepage is determined by the condition of discontinuous, but not all of discontinuous in rock mass play a role to rock mass seepage and the fracture network in rock mass does not take the place of seepage network. At least every facture is intersected by other fracture and boundary, or connected by the source/sink functions (like bore, drainage pipeline, etc). And it must be eventually connected to boundary and source/sink functions through inter-crossing, a flow path can be built [6]. Therefore, rock mass seepage network is a combination of some fractures which have water characteristics and are in the connected position of fracture network. It is systematically and thoroughly studied by Wang enzhi[7], Walter Wittke[8], Mo haihong[9], Lin dezhang[9], Jing lanru[10], Li haifeng[11], etc, to how to build seepage network. Three types of fracture surface domain are formed by discontinuous interaction cutting, namely block boundary type, terminal-shaped and isolated type. Later two types respectively correspond to dead end and isolated fractures in the seepage network. After the formation of seepage network, the water characteristics of every seepage face must be judged to distinguish water fracture and anti-water fracture, then seepage network will be adjusted accordingly. The specific details on how to generate three dimensional seepage network in fractured rock mass is shown in the literature [11].

2.2 The mesh generation of three dimensional fracture network

Three dimensional seepage network of fractured rock mass formed by search is set of fracture surface domains. These may be convex or concave, simply connect region and doubly connected region, their shape are more complex. It is difficult to finite element mesh generation. If these fracture surface domains will be assumed to qualification, it can be solved to how to generate the finite element mesh by the triangulation mesh generation of constrained delaunay technology [12].

If the mesh is formed by two dimensional triangulation mesh generation of constrained delaunay technology, fracture surface domain is carried on by constrained delaunay triangulation one by one, the geometry and topology information of intersected segments are constantly updated to sure the accordance of node. The specific implementation process as follows.

(1) To output fracture network according to each fracture surface domain in the format of planar straight line graph, PLSG for short;

(2) To select any one of fracture surface domain, establish its local coordinate system and process the coordinate transformation to transform the coordinate of constrained point and segment
from three dimensional to two, then form finite element mesh by two dimensional triangulation mesh generation of constrained delaunay technology, meanwhile need to update the geometry and topology information of its boundary segment;

(3) To the next fracture surface domain according to the inner-connected segment in fractures, and repeat step (2) to discrete mesh until all of fracture be subdivided.

If the mesh is formed by three dimensional triangulation mesh generation of constrained delaunay technology, the formed fracture network will be as a group. all of surface domain and its edge and vertex will be as constrained face, segment and point. To subdivide, three dimensional triangulation mesh generation of constrained delaunay technology is adopted directly, the mesh is got through retrieval. The specific implementation process as follows.

(1) To output three dimensional fracture network in the format of piecewise linear complex, PLC for short;

(2) To subdivide by three dimensional triangulation mesh generation of constrained delaunay technology;

(3) To select the mesh belonging to fracture surface domain from the grid, and form the finite element mesh which be needed to seepage analysis.

It needs to continually update the geometry and topology information of inner-connected segment by two dimensional triangulation mesh generation of constrained delaunay technology, and the selection of initial fracture surface domain has a significant effect on the final fracture mesh, it is difficult to be controlled for the element shape. Then all of fracture surface domain are treated uniformly by three dimensional triangulation mesh generation of constrained delaunay technology, the shape of element is a comparatively easy to be controlled. So the grid subdivision of three dimensional seepage network is got by three dimensional triangulation mesh generation of constrained delaunay technology.

3 The finite element equation of fracture network saturated/unsaturated, unsteady [4]

3.1 The fundamental assumption

Assume that the water of the fracture surface in the local coordinate system is two-dimensional unsteady flow, the water is exchanged through the links among fractures. Under the dimensional coordinate of fracture network, its seepage field is formed through the combination of two-dimensional flow of every fracture surface. The assumption is that.

(1) Fracture surface is isotropic and accords with Darcy’s law, namely

\[
\begin{align*}
  u_{x_1} &= -k_f \frac{\partial h}{\partial x_1} \\
  u_{y_1} &= -k_f \frac{\partial h}{\partial y_1}
\end{align*}
\]

(1)

Where \( k_f \) is the hydraulic conductivity of fracture, \( x_1, y_1 \) are the local coordinate system of fracture and \( h \) is hydraulic potential.
(2) The width of fracture is random value and can take hydraulic equivalent wide gap $a_h$, and saturated hydraulic conductivity coefficient of fracture is

$$k_f = \frac{g a_h^2}{12 \nu}$$

(2)

Where $g$ is the gravitational acceleration, and $\nu$ is the sports viscous coefficient of water.

(3) The relation in the degree of saturation in fracture $S$, unsaturated hydraulic conductivity coefficient $k_u$ and negative pore water pressure $P$ is treated as formula

$$s = e^{\eta h_c} (h_c \leq 0)$$

and

$$k_u = k e^{\eta h_c} (h_c \leq 0)$$

, or is represented,

$$k_u = k_f f_1(p)$$

$$S = f_2(p)$$

(3)

When the variable data of fracture width is lack, the following approximate formula is used,

$$f_1(p) = f_2(p) = e^{-5h_c^2} (p \leq 0)$$

(4)

(4) To ignore the water compressibility and not consider the effect of fracture deformation on hydraulic conductivity coefficient;

(5) To ignore the head loss caused by intersection in different groups of fracture.

3.2 The governing equation of fracture network saturated/unsaturated, unsteady

According to the water conservation equation and Darcy’s law, through derivation the governing equation of fracture network saturated/unsaturated, unsteady is obtained. Namely,

$$\frac{\partial}{\partial x_i} \left( k_u \frac{\partial h}{\partial x_i} \right) + \frac{\partial}{\partial y_j} \left( k_u \frac{\partial h}{\partial y_j} \right) + \frac{\partial S}{\partial t} = 0$$

(5)

Where $\frac{\partial S}{\partial t} = \frac{\partial S}{\partial h_c} \frac{\partial h}{\partial t}$, to unsaturated, $\frac{\partial S}{\partial h_c} = \frac{\partial S}{\partial h_c} = \eta \alpha h_c^{(a-1)} e^{\eta h_c}$.

3.3 The finite element equation of fracture network saturated/unsaturated, unsteady

Base on Galerkin method, the finite element equation of fracture network saturated/unsaturated, unsteady is established by the using the introduction of weight function, namely,

$$\sum_e \left\{ [A]^e \{h\}^e - [B]^e \left\{ \frac{\partial h}{\partial t} \right\}^e \right\} - \sum_e \{q\}^e = 0$$

(6)

Where

$$A^e_j = k_u \int_{-1}^{1} \left( \frac{\partial N_j}{\partial x_i} \frac{\partial N_i}{\partial x_i} + \frac{\partial N_j}{\partial y_j} \frac{\partial N_i}{\partial y_i} \right) J d\zeta d\eta$$

and

$$B^e_j = \gamma \int_{-1}^{1} N_j J d\zeta d\eta$$
3.4 Initial conditions

To all of nodes in three dimensional mesh, head are known when \( t = 0 \), namely

\[
h(x_i, y_i, z_i, t) = h(0, x_i, y_i, z_i)
\]

(7)

Where \( t \) is the node number.

Or can be written,

\[
h(n_i) = h_i(n_i, t)
\]

(8)

3.5 Boundary conditions

The unsteady boundary condition in fracture network hydraulic changes by time, the boundary condition can be classified into:

1. The boundary condition of given head

\[
h(n_i, t) = h_i(n_i, t)
\]

(9)

2. The boundary condition that the given head is equal to the boundary

The case of overflow is such boundary condition, when the infiltration caused by rainfall is more than the possible infiltration intensity of surface fracture, each node in the ground belongs to such condition, namely,

\[
h(n_i) =
\]

(10)

3. The boundary condition of given velocity

\[
\frac{\partial h}{\partial n} = V_n
\]

(11)

When the infiltration velocity of surface fracture is smaller than its intensity in rainfall, such condition is preferred.

4 The example

Based on the above algorithm, a program 3D-Network-Seepage.f90 is developed to three-dimensional seepage network searching of fractured rock and saturated/unsaturated, unsteady-state seepage analysis. Its correctness is verified through the test example dawn up by Y.I.Kim in the paper [13, 14]. The test result of T1-5 is verified in paper, namely the boundary condition is \( h=71.76\text{cm}, H=80.52\text{cm} \). The initial condition is that there is no water in fracture network and it is presented by a negative pressure value \( p/\gamma = -1.5m \). The time step \( \Delta t \) is 0.2S and 40 time steps are calculated. The detailed results are described as follows.

4.1 Total head analysis

It is known that the total head of every measuring point changes from negative pressure to positive gradually and achieves the stable state after \( t=1.2 \text{ s} \), no changing (see fig1). It is known from table 1 that
the calculated value basically agrees well with measured value with the largest difference less than 1.07%, which indicates the method can generate objective and correct seepage network and accurately reflect the motion law of water flow in fracture network.

![Image of Time-total head changes history curve of all test points](image)

Figure. 1 Time-total head changes history curve of all test points

<table>
<thead>
<tr>
<th>Measuring point</th>
<th>The first step (t=0.2s,unsteady)</th>
<th>The fifth step (t=1.0s,unsteady)</th>
<th>The sixth step (t=1.0s,steady)</th>
<th>errors (%)</th>
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<td>71.88</td>
<td>71.88</td>
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</table>
4.2 Pressure head analysis

The contours of every step pressure head in fracture network are shown in Fig 2, no colour is the negative domain, namely no water domain. It is known from Fig 2 that the seepage state of every node in fracture network exchanges from unsaturated to saturated, namely no water to water, and all reach the saturated state when time is 1.2 s. The water flow from the left and right sides to the central regions of network separately, and to every no water domain, dead end, etc, through the cross node successively, in finally fills the total fracture network and makes it saturation. The water change law reflected by contours can more accurately reproduce the flow trend and motion law of actual water in fracture network.

5 Conclusions

Several key problems are researched systematically on fractured rock mass unsaturated seepage by discrete fracture network seepage model in the paper. To the build problem of seepage network,
three-dimensional fractured rock flow network search algorithm was formed by improving the existing three-dimensional block cutting algorithm and increasing the judgment of waterproof properties of fracture, etc. The problem of mesh generation caused by irregular fracture was solved by constrained Delaunay triangulation technique that seemed these fractures as constrained conditions. Then based on relationship function between unsaturated seepage parameter and unsaturated seepage by Prof. Zhang, the finite element equation was established by Galerkin method on unsaturated/unsaturated unsteady seepage analysis of fractured rock network. It was proved that the direction of water flow in fracture network was in accordance with realistic flow direction, and the calculated values of measuring point head water was in well keeping with measured values in steady state, the maximum error of less than 1.07%, which showed that the method could produce objective and accurate percolation network, but also could generate a reasonable computational mesh; and the calculations reflecting the changes of water flow could accurately reproduce the actual fracture network trends in water flow and movement rules.

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ANALYSIS ON APPROPRIATE CALCULATED METHOD OF THE SUPPORT STRUCTURE OF DEEP METRO EXCAVATIONS IN WUXI AREA

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Considering the unique geological environment of Wuxi area, introduce current situation of the excavation support structure in this area and explore the appropriate calculated method for the support structure of excavations in there. Combining with the monitoring data of the Public Square Station of Wuxi rail transit No.1, analyse the law of the diaphragm wall deformation resulted by many factors, determine the insertion ratio of diaphragm wall of the conventional subway station in Wuxi region, discuss the rationality of total stress approach for lateral earth pressure and effective stress approach for lateral earth pressure. Finally, the appropriate calculated method is suggested. This research can provide a reference for the design of the metro excavation engineering in Wuxi area in the future.

1 Introduction

In recent years, with the construction of subway engineering, many deep and complex excavations have appeared in Wuxi area. The construction of underground engineering has a great challenge to the design, construction and monitoring technology in this area. There is no technical code for excavation engineering in Wuxi area, and some accidents, like pit collapsing, retaining wall having too large deformation, pit leaking, have took place during construction. So it is very necessary to explore the appropriate calculated method of the support structure of deep excavations for the geological environment in Wuxi area. Diaphragm wall is the main retaining structure of the deep metro excavations in this region. Engineering experience shows that the cost of diaphragm wall is about 30% of the total cost of metro station construction, so appropriate calculated method is very important for guiding the design of diaphragm wall, it can not only make diaphragm wall meet the deformation and load carrying capability, but also reduce the project cost.
2 The status of the retaining structure of the deep metro excavations in Wuxi area

At present, there are several kinds of retaining structure in Wuxi area. The first one is gravity cement-soil wall, which is mainly used in the excavations whose depths are less than 5 meters. The second one is compound soil nailing wall, which is mainly used in the excavations whose depths are about 10 to 15 meters. The third one is contiguous bored pile wall with strut or pre-stressed anchor cable, which is used in the excavations whose depths are greater than 6 meters. The last one is diaphragm wall or retaining wall formed by SMW method with strut or anchor rod, which is used in the excavations whose depths are greater than 10 meters and the surroundings around excavations are complex [1].

There are totally 19 underground stations in Wuxi rail transit No.1, including 18 stations which use diaphragm wall with steel support or steel reinforced concrete support as their retaining structure. Considering there is no experience of design and construction about metro excavations, it is very necessary to explore the appropriate calculated method of the support structure of deep excavations in Wuxi area. This paper mainly discusses suitable calculated method of diaphragm wall to provide the reference for the subway stations designers in this region.

3 The calculation theory of the deep metro excavations’ diaphragm wall in Wuxi area

The experiment and theoretical research show that method of vertical beam on elastic foundation is appropriate to calculate the internal force of diaphragm wall, because this method can reflect the characteristic of diaphragm wall, and can consider many cases, such as no anchor, with single anchor and several anchors. The key of using the method of vertical beam on elastic foundation is to determine the resistance coefficient of the ground, which is usually determined by “m method”. This is the preferred method in the engineering[2]. In order to research the law of deformation of diaphragm wall, this paper takes “m method” to calculate and analyse the displacement and internal force of diaphragm wall. In addition, for the deep excavations which have obvious effect of space, method of plate on elastic foundation can be used to calculate the displacement and internal force of diaphragm wall and continuum finite element method can be used in the complicated excavation engineering [3].

4 Influence factors on the inner force and deformation of the deep metro excavations’ diaphragm wall in Wuxi area

This paper calculates the excavation working condition of the Public Square Station of Wuxi No.1 subway to analyse the influence on the inner force and deformation of diaphragm wall by several major factors. The Public Square Station is constructed by open excavation order method. The depth of the excavations is about 15 meters and the surroundings around excavations are simple. The related information about this station can see Literature 4.

4.1 The influence on the inner force and deformation of diaphragm wall by diaphragm wall’s thickness

Through the comparison of the calculation results under various thickness of diaphragm wall, we can see from figure 1 that the maximum horizontal displacement of the wall on the three surveying slant holes’ sections which are CX6, CX10 and CX12 have a same change trend that the maximum horizontal displacement of diaphragm wall is gradually decreasing along with the increase of the thickness. The relationship between the thickness of diaphragm wall and the maximum horizontal displacement of diaphragm wall can be described by the equation of

\[ y = 53.27 - 0.17x + 0.0000202x^2 \]

, in
which \( x \) is the thickness of diaphragm wall and \( y \) is the maximum horizontal displacement of diaphragm wall.

![Diagram](image1)

**Figure 1** The relationship between the thickness of diaphragm wall and its largest horizontal displacement.

![Diagram](image2)

**Figure 2** The relationship between the thickness of diaphragm wall and its largest bending moment.

We can see from figure 2 that the largest bending moment of diaphragm wall is gradually increasing along with the increase of the thickness. The relationship between the thickness of diaphragm wall and the largest bending moment of diaphragm wall can be described by the equation \( y = 155.07 + 1.17x \), in which \( x \) is the thickness of diaphragm wall and \( y \) is the largest bending moment of diaphragm wall.

It is clear that increasing the thickness of diaphragm wall will reduce its displacement, but will cause the increase of the moment of diaphragm wall. Besides, increasing thickness will make construction cost increase, so the thickness of the wall is not the bigger the better, but is decided by combining with several key factors. For the excavations which have long and narrow shape and have simple surroundings, according to the control requirements of the displacement of diaphragm wall, we can reference the equations given in this paper to preliminary design the thickness of diaphragm wall.

4.2 The influence on the inner force and deformation of diaphragm wall by support interval

The steel supports in metro excavations are generally away 3 meters from each other. If the support interval is too large, the deformation of diaphragm wall will become excessive and the construction work will become inconvenient. In order to analyse the influence on diaphragm wall by support interval, we calculate the inner force and deformation of diaphragm wall with different support interval which range from 2 meters to 5 meters.

![Diagram](image3)

**Figure 3** The relationship between support interval and the largest horizontal displacement of diaphragm wall.

![Diagram](image4)

**Figure 4** The relationship between support interval and the largest bending moment of diaphragm wall.
From figure 3 and figure 4 we can see that both the maximum horizontal displacement of the wall and the largest bending moment of wall are gradually increasing along with the increase of support interval. The maximum horizontal displacement of the wall increase almost linearly along with the increasing of support interval and the relationship between them can be described by the equation of \( y=17.08 + 0.89x \), in which \( x \) is support interval and \( y \) is the maximum horizontal displacement of the diaphragm wall. The relationship between support interval and the largest bending moment of diaphragm wall can be described by the equation of \( y=637.30 - 47.26x - 1.29x^2 \), in which \( x \) is support interval and \( y \) is the largest bending moment of diaphragm wall. We can see that support interval have a major influence to the inner force and deformation of diaphragm wall. So we have to consider the security level of foundation pit and combine several factors, such as the depth of foundation pit, excavation scale, the complexity of surrounding environment, to decorate support reasonably. For the excavations which have long and narrow shape and have simple surroundings, we can reference the equations given in this paper to preliminary design support interval.

### 4.3 The influence on the inner force and deformation of diaphragm wall by the insertion ratio of diaphragm wall

Controlling the insertion ratio of diaphragm wall in deep excavations is to ensure that the stability of the wall. In actual design of deep foundation pit, the insertion ratio of diaphragm wall is controlled for economic reasons. For the excavations having about 15 meters depth in Shanghai region, the stability of the foundation pit can be guaranteed when the insertion ratio of diaphragm wall is controlled within 0.8 to 1.2. For Wuxi area, in order to find the suitable insert depth of the wall, we calculate the inner force and deformation of diaphragm wall by changing the insertion ratio of it and keeping other conditions remain unchanged.

From figure 5 and figure 6 we can see that both the maximum horizontal displacement of the wall and the largest bending moment of diaphragm wall are gradually decreasing along with the increase of the insertion ratio of diaphragm wall when the insertion ratio is less than 0.6. In this condition, increasing the insertion ratio is favourable to control the force and deformation of diaphragm wall and to increase the stability of the foundation pit. But when the insertion ratio is more than 0.6 later, increasing the insertion ratio almost has no influence on the horizontal displacement and bending moment of diaphragm wall. At this time, by increasing the insertion ratio to adjust the inner force and deformation of diaphragm wall is not economical. Through the above analysis, taking the insertion ratio...
of diaphragm wall as 0.6 is reasonable for the metro excavations whose depths are less than 15 meters and surroundings are simple in Wuxi area.

5 Analysis on appropriate calculated method of the deep metro excavations’ support structure in Wuxi area

In this paper, we compare the horizontal displacement of diaphragm wall on three surveying slant holes sections which are CX6, CX10 and CX12 with the calculated result by the method of total stress approach for lateral earth pressure and effective stress approach for lateral earth pressure. The methods of selecting m value, which reference the technical code for excavation engineering[5]and the building foundation pit engineering technical specifications[6], include the look-up table method in the Shanghai standard, the formula method in the national standard and the look-up table method in the national standard. Comparing the measured value of horizontal displacement of diaphragm wall and the calculated value by different methods, the result is shown in figure 7.

In pictures of figure 7, series No.1 represents the value calculated by the method of deciding m value using the table in the Shanghai standard and the method of total stress approach for lateral earth pressure, series No.2 represents the value calculated by the method of deciding m value using the formula in the national standard and the method of total stress approach for lateral earth pressure, series No.3 represents the value calculated by the method of deciding m value using the table in the national standard and the method of total stress approach for lateral earth pressure, series No.4 represents the value calculated by the method of deciding m value using the table in the Shanghai standard and the method of effective stress approach for lateral earth pressure, series No.5 represents the value calculated by the method of deciding m value using the formula in the national standard and the method of effective stress approach for lateral earth pressure, series No.6 represents the value calculated by the method of deciding m value using the table in the national standard and the method of effective stress approach for lateral earth pressure.
Figure 7 the comparison of measured deformation of diaphragm wall with the calculated deformation.

From figure 7 we can see that the deformation of diaphragm wall calculated by the method of deciding \( m \) value using the formula in the national standard and the method of effective stress approach for lateral earth pressure is more closed to the measured value than any other methods.

When we using the method of total stress approach for lateral earth pressure, the deformation of diaphragm wall calculated by the method of deciding \( m \) value using the table in the Shanghai standard, the method of deciding \( m \) value using the formula in the national standard and the method of deciding \( m \) value using the table in the national standard have biases with the measured value. The calculated results by the first two kind of method are both less than the measured value. The maximal displacement calculated by the last method is closed to the measured value, but the position of maximal displacement is lower than the measured value a lot.

When we use the method of effective stress approach for lateral earth pressure, the calculated result by the method of deciding \( m \) value using the table in the national standard is less than the measured
value and the calculated result by the method of deciding m value using the table in the Shanghai standard is larger than the measured value.

To sum up, for the excavations which have long and narrow shape and have simple surroundings in Wuxi area, we suggest that the method of deciding m value using the formula in the national standard and the method of effective stress approach for lateral earth pressure can be used to preliminary estimate the inner force and deformation of excavations.

6 Conclusions

(1) The relationship between the thickness of diaphragm wall and the maximum horizontal displacement of it can be described by equation of \( y = 53.27 - 0.17x + 0.0000202x^2 \), in which \( x \) is the thickness of diaphragm wall and \( y \) is the maximum horizontal displacement of diaphragm wall. The relationship between the thickness of diaphragm wall and the largest bending moment of it can be described by equation of \( y = -155.07 + 1.17x \), in which \( x \) is the thickness of diaphragm wall and \( y \) is the largest bending moment of diaphragm wall. For the excavations which have long and narrow shape and have simple surroundings, we can reference the equations given in this paper to preliminary design the thickness of diaphragm wall.

(2) The relationship between the maximum horizontal displacement of the diaphragm wall and support interval can be described by the equation of \( y = 17.08 + 0.89x \), in which \( x \) is support interval and \( y \) is the maximum horizontal displacement of the diaphragm wall. The relationship between support interval and the largest bending moment of diaphragm wall can be described by the equation of \( y = 637.30 - 47.26x - 1.29x^2 \), in which \( x \) is support interval and \( y \) is the largest bending moment of diaphragm wall. For the excavations which have long and narrow shape and have simple surroundings, we can reference the equations given in this paper to preliminary design support interval.

(3) Taking the insertion ratio of diaphragm wall as 0.6 is reasonable for the metro excavations whose depths are less than 15 meters and surroundings are simple in Wuxi area.

(4) We suggest that the method of deciding m value using the formula in the national standard and the method of effective stress approach for lateral earth pressure can be used to preliminary estimate the inner force and deformation of excavations.

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MODERN MINING CONCEPT AND FILLING TECHNIQUE

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Abstract: The waste rock, tailings and smelter slag in the process of the exploitation of mineral resources accounted the industrial solid wastes for about 85% in China based on traditional mining pattern. A large number of the solid waste piled up on the ground in mine can cause serious pollutions, and may induce mudflows and tailings dam-break accidents. As the supporting technology of green mining, the solid waste backfill is the most effective way to solve the discharge of mine waste. Undoubtedly, the filling is endowed with the function of eliminating surface subsidence caused by mining, improving the mining stress environment, operating in low dilution and loss, increasing the rate of comprehensive utilization of resource, mining rich ore and reserving lean ore for the prospective resources protection, reducing the waste rock, tailings and other solid waste emissions, even eliminating the terminal treatment of mining to achieve waste-less-mining, and also with the function of adapting to a variety of difficult-to-mine ore-bodies. Furthermore, principles and technical support for implementing green mining and research directions of filling technique are discussed in this paper.

1. Introduction

21st century is the period with rapid development and growth of modern science and industry, the scale of development and utilization of mineral resources has never appeared before. The mining depth of metallic mineral resources is increasing, the exploitation is more and more difficult, the object is gradually shifting to deep orebody in heavily stressed environment and “three unders” difficult mining orebody under complex conditions. Mining industry faces problems of geologic disasters such as deep rock burst and rock burst, ecological conditions such as ground depression and surface subsidence, technological economy issues such as production capacity and production costs. With the intensification of world mining industry, a large number of industrial emissions such as waste water, wastes and exhaust gas lead to deterioration of ecological environment. A growing awareness of the importance of acceptable mine and comprehensive utilization of resources and an urgent need to implement green mining. Green mining pattern is to minimize waste and emissions, improve the comprehensive utilization of resources, reduce or eliminate the negative impact of mineral resources development. Green mining pattern is the scientific outlook on development embodied in mining business, no doubt, the solid waste backfill is the main support of green mining technology.

2 Traditional mining pattern

It is traditionally considered that mining is the front-end process of mineral resources development, ecological restoration and environmental management of mining area are the end processes. In the process of ore mining, people often focus on economic activity, but ignore the serious negative impact to natural environment. People only do the end process in case of serious ecological damage or environmental pollution even significant hazards, but seldom solve mining and environmental
protection in accordance with mining and the ecological environment in harmony as a keystone of the whole process (D.V.Boger, 1998).

Clearly, the core limitation of traditional mining-concept is the inconsistency between ore mining(front) and environmental management(end), which leads to segmentary processing and a facile solution. Thus, the development over the years has caused series of negative impact on natural ecosystem and economic, such as a lot of land is damaged, large amounts of solid waste piled up on the ground and a large number of goaf existed underground, which has become the major issues to be solved currently, and include the following aspects.

2.1 Waste of resources and insufficient resources

China has found 171 kinds of mineral resources, 158 species have been identified the storage capacity; it has 18,000 minerals and more than 7,000 large and medium sized mineral deposits. Statistics show that the utilization extent of advantage identify mineral resource is better than the disadvantage mineral resources, there are many contradictions and problems between the utilization extent of mineral resource and the principia of allocation of mineral resource market. The excessive exploitation of advantage resources just for immediate interest, which results high utilization of some advantage mineral resources, acute shortage of resource in support and leads advantage status between the beetle and the block. The mining industry excavates ore rock of 28 billion tons per year, the transport capacity is more than erosion of all the rivers on the planet, only copper and other non-ferrous metal smelting can release as many as 6 million each year. At present, our total mining scale is ranking third in the world, the annual extraction is over 50 million tons. China has about 8500 mining enterprises of the whole people now, which total output value is more than 65 billion, is about 7% of the total industrial output value, solid raw material such as non-ferrous, ferrous and coal occupy a major proportion. More than 90% of the energy, 75% to 80% of industrial raw materials are from mineral raw materials, mining and mineral processing industry output value accounting for more than 30% of GDP, supporting more than 70% of the function in various departments of national economy. Mineral resources as raw materials in industrial output value related to the gross national product accounted for 1/3 to 1/2. In recent years, import and export volume of minerals and related raw materials has accounted for the proportion of foreign trade import and export volume reached 25%. Then, take non-ferrous metal industry for example, in our existing 124 industries, 113 industries use the non-ferrous metals, the linearly dependent coefficient of the consumption level of copper, aluminum, zinc and other major non-ferrous mineral resource and GDP is over 0.9, its production capacity and GDP is also exceeded 0.9.

According to data conducted by some authorized institutions, there are 9 nonferrous metals in our country, and during the 10 years from 1995 to 2005, the recovery increased from 38% up to nearly 64%, however, which is still lower 20% than other developed nations. At present, the operating factor in mineral resources of our nation is about 35%. The deposite that produce nonferrous metal are mainly multi-metal ones, in which many useful mineral resources coexist. Their overall recovery draw great emphasis from people. Some developed countries like America and Japan hold high recoveries from 76% to 90% in multi-metal mines producing copper, lead, zinc and nickel. The best 9 copper concentrators in USA have reached a highest recovery about 88-91% by recovering some elements in copper, gold, silver, aluminum and molybdenum. The recovery of the multi-metal mine in Japan is 85% by recovering lead, copper, zinc, gold, silver and barite. Canada recovers caesium, beryllium, gallium from pegmatite ore. The nations above have reached high overall recoveries, whereas, lower recovery
exists in our country. According to research on 1845 important mines in which 2% mines get a high recovery of more than 70%, less than 15% have more than 50% and mines with recoveries less than 25% account for 75%. In 246 mines with big or middle scales, 32.1% of them have never made use of the useful ingredients of mineral resources and these useful ingredients are removed along with gangue, which causes great waste of natural resources, and increase the bad influence on environment. In contrast with developed countries, our technology in recovery are in low level and we need put in effort in many researches and practical works on exploitation of resources (R. Cowling, 1998).

At present, resource waste still exists in our nation, besides, we face the lack of the resources. On the whole, China is rich in resources, which stands in the third place in the world scale, but the per capita share of resources is lower than the average level of the world, and ranks in the 80th place. Only 20 kinds of 45 minerals can meet the need of the domestic production, and we need 0.2 billion more tons of metallic minerals. According to the investigation, we need less resources that is rich in China, but we need more what we lack. Some authorities estimate that 24 of 45 main minerals can meet the need before 2010, and only 6 of them can be the minerals that we do not lack. It is estimated that in the future 20 years, the demand of petroleum, natural gas, copper and aluminum is 2 to 5 times of the reserve volume. China will need 3 billion tons of metal, 0.6 billion tons of copper and 0.1 billion tons of aluminum. Taking the non-ferrous metal as an example, development needs more amount of resources, and many mines are so complex and hidden that the exloitation is very hard to conduct. What is worse, the fund on the geologic exploration is less than before, the speed of increase on reserve volume is slow. The available volume accounts for 30% in the need of resources. It is estimated that we need more than 5 million tons of non-ferrous metal, and we have to import copper, diamond and platinum after 2010. Some minerals such as aluminium, zinc, nickel, gold and silver will have to be import in a long time. Many companies on mining founded in about 50th in 20 century, and they have been exploited about several decades, so more than half of them can not work for a long time. Further more, some companies lack of funds and modern devices, they are not able to exploit the resources in the deeper land. The two reasons are also the main spectacle in development of companies. It is estimated that 110 mines are closed due to the lack of minerals and 440 will face the same problem. For example, till now, nearly a hundred of non-ferrous metal mines have closed according to the requirement of government, in 2020, 20% of mines will exist[7, 8]. It is clear that the development of national economy will be faster in the future 15 to 20 years, the balance between supply and demand will not be reached, however, in order to build a well-off society in an all-round way, the more mineral resource will be needed, so the situation of the exploitation of mineral resources is till tough, and we need put in more efforts in it.

2.2 The damage of surface collapse

Before exploitation, the environment of mining areas is comfort with the around area, however, after exploitation, the environment is destroyed and much different. The destroyed land is between fourteen thousand and twenty thousand square kilometers, and increasing two hundred square kilometers per year. According to the statistic to 1173 stated owned big-and-middle-sized mining areas, 68.89 percent is underground mining, subsidence area is 84201 hectare which covers 39.57 percent of the total damaged mining area. More than 30 cities suffer the damage of mining collapse. There are almost 50 mining areas have serious collapsed area according to the rough statistic. Such as Yangquan where collapsed area more than 60 square kilometers, Jiaozuo almost 52 square kilometers, Fushun 30 square kilometers, and Pingdingshan 20 square kilometers. Averagely ten thousand tons coal exploitation is company with more than 2000 square meters collapse area.
(1) Damage to land form

Movement basin, surface crack and sunken pit destroy the surface configuration and stability. Surface movement basin which is also called low-lying land is formed by land collapse. Such kind of terrain light caused difficulty to tillage and irrigation, even caused surface ponding, which block tillage and makes land loss its value. Surface cracks destroy the continuation of land, bring difficulty to tillage and serious soil erosion, also bring potential geological disaster. Sometimes subsidence could destroy ground aquifer and decline the water table and bring drought disaster. Mining collapse will destroy ecological balance even change the type of ecological system. mining collapse destroy forest, grassland, natural vegetation, and terrain, water and air polluted caused by mining collapse deteriorates the ecological environment. The worsen ecological environment lead to new natural disaster such as soil erosion, land salinize, swampiness, desertification, all of such problem will imperil society development, and then lead to new natural problem and economic problem.

(2) Destroy to land resource

Mined out area breakdown change the zero depth, destroy the food stuff of land, and formed wetland or drought land, which makes soil acidification and lost its own value. Until then end of 1991, the total mined out area breakdown reached one million and five hundred thousand acres. According to the statistic, averagely ten thousand tons coal exploitation is company with more than three acres collapse area and even fifteen point one acres, and half of the collapsed land is the plough land. The exploitation is one hundred million per year and the total collapsed land is increasing thirty thousand acres, and the collapsed plough land is increasing one hundred and sixty-five acres. Mountain and hilly land is north-west, north-east, and north China has no obvious change, but land collapse formed funnel sink or crack. In some area, it may forms big crack, landslide and ground water exhausted which increase the development of desertification. Such as Xuzhou Mining Bureau, the collapsed plough land is about more than eighty hundred acres, 25 percent of which is stagnant water areas. And in Huaibei Bureau, the collapsed plough land increase 50 acres every year, and 35 percent is stagnant water area, the depth of water is more than ten meters. In these areas, farm land is destroyed badly and creased to produce any food. So we can see that the damage of mining collapse to land resource is substantial and caused series of social problems.

(3) Destruction of water resource

Water drainage and natural drainage to the water level of coal from water fracture in the traditional process of coal mining has destroyed the groundwater resources and dried up underground water. The fall of the gob make the ground subsidence of rivers, canals, ponds, motor-pumped well dry up and deformation and the mining area residents face water difficulty. Grant collapse not only changes the conditions of surface water and underground water runoff, and also the water quality, water pollution. The existence of ground collapse has destroyed the underground aquifer structure, changed the groundwater flow direction, the underground water level dropped substantially, the water sources dried up, and irrigated farmland would be easily changed into early. The change of surface and underground system, the industrial and agricultural production water and water have been affected. The surface movement basin often makes the surface of the ground mining area throughout the low water, water depth a few meters to more than 10 m, some local water depth and even reaches to 15 m and above. Because of massive mining sewage and waste water discharge and contaminated, surface pounding often become the dirt, germs the collection. And the ground water pollution and often put over alley, rock fracture along goaf flow backward into the ground, underground aquifer contaminated, form vicious circulation. The mining area and its near a water, water pollution, and biological hazards to
human beings is serious; Make crops low production, and even no production; Make fish and large death aquatic organisms, aquatic breeding to bring evil bad luck; Make human body produce acute or chronic poisoning, or become the growth of various epidemics, bring threat to the human health and safety biological survival.

(4) Destruction of the building

There are three kinds of deformations, they are surface deformation caused by underground mining (sinking, the tilt, bending, and twisted), the horizontal mobile deformation (level mobile, stretching and compression deformation), and the surface of the shear deformation. The surface movement and deformation destroy buildings and foundation of the balance between preliminary state, with focus on and balance to establish, make the building to generate additional stress, and lead to building the distortion, until the damage and destruction. The earth surface subsidence also has serious harm to the normal transport of highway and railway, like the road surface subsidence, and even water, railway roadbed collapse, railway bending or impending track. Surface subsidence of the damage to bridge, often than road or rail itself damage because bridge integrity is strong, stability is strict, length of the cost is high, so once it's hard to repair the damaged, and even completely ruined, cause great losses. In addition, the surface movement caused by mining will make transmission, line tower skewed, and the line relaxation degree change. For high voltage power transmission lines, it not only falls easily destroyed by itself and human and animals around life were a serious threat. Big range will surface subsidence will seriously affect the dam, horizontal displacement can destroy the integrity of the dam body, make appear tensile fracture belt and fracture zone, so as to reduce the strength of the dam and seepage control ability. The influences of the sink reduce the top elevation, reduce or lose the ability to intercept the flood peak flood season.

Recent years, as our country mining industry especially individual mining development, our country mining subsidence disaster occurred frequently. The statistics from the provinces, it caused most of the collapse in empty individual industry, the second is the collective enterprise, and then is state-owned enterprises. Nationally, goaf collapse and mining quantity the relationship is linear, mining, the more the more serious collapse, to the society economy system and natural ecological environment of the harm caused by bigger also. Therefore, we must use the ecological and economic point of view to handle and coordinate resources development and environmental protection to the contradiction, make resources development and environmental protection coordination development, in order to obtain the best economic benefit and social benefit and environmental benefits.

2.3 Environmental damage caused by waste emissions

By the end of 2005, China had several of mine enterprise 183069. According to not complete count, 1949-the end of 2005, the national various of mine output all kinds of ore waste stone 16.23 billion t, 3.56 billion t of coal, iron ore waste stone 9.4 billion t, 2.5 billion t of nonferrous metal waste stone, stone 460 million t waste gold mines, chemical ore waste stone 300 million t. Our country's existing DAMS 12718 seats, including the tailings for 1526 seat under construction, accounting for 12% of the total, had closed library tailings 1024 seats, or 8% of the total, by the year 2007, the total for 8.046 billion tons of tailings accumulation. Only in 2007, the national tailings discharge close to 1 billion tons. In addition, the stockpiling of fly ash 1.2 billion t. The development and utilization of mineral resource produces in the process of coal gangue, waste stone, fly ash and smelting slag has become China's emissions of the largest industrial waste, accounting for about 85% of the total amount of solid waste. Metal mine tailings is in process of the main waste produced, the metal mine emissions
stored up to 280 million tons a year with tailings growth, covers an area of 140000 mu, every year, covering an area of about 2000 new mu, and the utilization rate of tailings is only 1.5%.

Coal gangue is coal production and processing produces in the process of solid waste, the annual emissions equivalent to that of coal output 8 to 20%. According to concerning sectional statistic, there are more than 1,500 coals national large and medium-sized coal mine gangue piles at present. Coal gangue stored up more than 3.4 billion tons, covers an area of 200000 mu, and with the speed of 200 million tons a year growth, as one of the largest industrial solid wastes emissions in China. Each year, it takes 1 billion RMB on disposal of coal gangue, but with utilization rate only 40% or so. Coal gangue, waste stone, backfilling materials, waste water and waste long-term stockpiling, take up much of the land, and cause spontaneous combustion, air pollution and underground water quality.

2.4 Potential safety hazard

Gob area may cause many dangerous things happen, such as subsidence, earthquake, collapse, landslide and water inrush. There are 30 cities in which subsidence happened, and 25 of them were serious accidents. Nearly 200 serious subsidence accidents happened in our country, which cause great loss. For instance, Yunnan Tin Group AG has a history of nearly 100 years in mining, and it has many way of mining, such as open stope method, shrinkage method, caving method, board and pillar method and flashing system. The construction of mine area is so complex that potential safety hazard exists during mining. Besides, it is hard to work for workers in the place filled of oxidized ore and sulphide ore. The biggest danger in mining is destabilization in gob area. In 2004, a serious accident cause by subsidence happened in a gypsum area in Feng cheng of Shandong province. According to statistics, more than 180 accidents happened in 20 provinces in our country, which caused a loss of more than 50 billion.

Solid waste stacked on the land surface may easily cause debris flow and dam break. According to the investigation in 2004, many of tailing areas are dangerous.1035 areas were measured with degree of safety, in which the most dangerous ones account for 5.9%, the lower and dangerous ones account for 6.7% and the least dangerous one account for 25.3%. Many accidents, such as water pollution and debris flow, which are caused by solid waste are rearful. During the process of exploiting, sulphide in the solid waste and ettle oxidate, and then make the water acid. The acid water consists of Fe, Mn, Ca, Mg, Al, Cu, Zn, Pb, As. Ground water is polluted by the waste water that even influences the whole ecological system. After analysis of polluted water in 5 copper ores conducted by Nanchang institute of nonferrous metallurgy design Institute, it was found that river water mixed with waste water from copper ore was used to irrigate the farm land, which caused the content of copper to increase 16.5 times compared with the normal field. Nearly 7000 Mu of farm land were polluted and less rice was produced. Many accidents happened in different places of our country, which led to terrible results.

3 Modern theories of mining and filling mining

Sustainable development is the epoch making developmental theory and one of the most fundamentalist changes in human ideas of the 20th century. It includes ecological, economic and social sustainability, respectively being the foundation, the key and the aim of the sustainable development in our country. Ecological sustainability is based on the limited ecological condition and means taking appropriate measures in environmental resources and capacity to promote the normal functions of ecological systems. It requires the effective utilization and recycling of resources. In 1989 the world
committee of environment and development put forward the plan of clean production and ISO14000 environmental management series standards. These two plans of action have a positive effect on the sustainable development of mining. Clean production has become an important strategic measure in the sustainable development of mining. According to the ideas of sustainable development, Academician Qian Minggao and his research team proposed the green mining theory and built the green technological framework at the beginning of this century. Green mining mode is directed against the environmental problems caused by mining and it treats coal, gas, underground water, earth and all other usable resources from the perspective of the generalized resources. Green mining intends to reduce the harm done to the environment by mining, a technique which integrates both the resources and environment is needed, its purpose is to prevent or reduce as far as possible the harm done to the environment and achieve the economic, environment and social benefits. The technical framework of green mining includes water-preservation in mining areas, coal mining to retard surface subsidence, simultaneous extraction of coal and coal-bed methane, reduction of rock waste, underground coal gasification, and others. On the technical side, the focus should be transferred from the influence of the strata movement on the coal face to its influence on Surface Subsidence and the rules of gas and liquid movements in the strata, and provide theoretical support to water-preservation in mining areas, coal mining to retard surface subsidence, simultaneous extraction of coal and coal-bed methane, reduction of rock waste. On the policy side, the research should concentrate on the different green mining modes, the system of economic evaluation and its relation to the production costs, the economic characteristics of coal mining resources and to provide advice to government policy. To ensure the sustainable development of coal mining companies, the government should give green mining support in the respects of policy and taxes.

Above all, the green mining mode can reduce as far as possible the production and emission of wastes, improve the efficiency of using resources and alleviate or prevent the harmful effects of mining. It is the embodiment of scientific developmental ideas in coal mining. To achieve green mining, the mining companies should integrate usage of resources, humane environment, ecological environment and economic factors, form an industrial system and solve problems from that perspective.

The basic meanings of green mining include:

- Improving the efficiency of using resources and recycling the low-grade ore. Considering the non-renewable features of resources, recycling and protective exploitation should be combined.
- Making the minimum wastes, which means reducing the emission of wastes as far as possible to lessen the harm done to the environment.
- Using the wastes as resources, recycling the wastes and protecting them as second-hand resources.
- Developing high technological mining methods to meet the needs of green mining.

3.1 The mode of green mining and filling methods

According to the aims of green mining, it is clear that the modern green mining need to appeal to three requirements, which are overall resource utilization, the least waste emission and avoiding the destruction of the earth. Solid waste cut and fill stopping is the main method of green mining, which help to avoid the danger of ground depression and protect the environment. Furthermore, it can reduce the solid waste and even reach the goal of non-waste mining (R.Cowling, 1998).
3.2 The REDD function of the filling technology

At present, cut and fill method has been used widely with much develop progress. In reality, the method has been used in many fields except mining (P. Farsangi et al., 1996). Because the overall resource utilization and environment protection are the basic requirements in reaching the hope of sustainable development of national economy (Yilmaz et al., 2003). It can be achieved to found the mine without waste and realize green mining (Amaratunga et al., 1997). There are few typical mines without any waste. In Germany, two mines are famous for their advanced methods on exploitation of waste. One is Ground Lead Zinic Mine, which fill the route with tail and gallet, and does not produce waste. Another examples of successful is in Germany. The coal from it is cleaned in coal washery, then the cleaned coal is used for power generation, and finally, the waste from power station and coal washery is used to fill the gob area. Besides, some mines in domestic and foreign scenic spots work without waste, because these mines pay more attention to the backfilling. Reusing industrial solid waste is the most important part for modern mines that should try their best protect environment.

4. Principles and technical support of green mining

4.1 Guidelines and technical principles

In order to realize the goal of green mining, it should be carried out to control the environment pollution and protect the ecological environment, and to implement the discipline of recycling that means developing green mining methods without damaging the ecological environment around mining area and non-waste or less waste techniques with less waste, and extracting firstly the valuable metal and useful energy, then putting the rest into other uses such as building, at last operating innocent treatment.

Government should create a system that aims at protect the environment and prevent pollution. The system should include many aspects such as proportion of extraction, rate of ore dilution, recovery, and overall utilization rate.

4.2 Mining design

Enterprises exploiting mineral resources should put more emphasis on two aspects. The first one is formulation and designation about the overall exploitation on mineral resources, including the exploitation of resources, the protection on natural environment, the preservation of water and earth, and the recycling of the land discarded. The evaluation on influence at environment is another one. The developed techniques used in mining should be adopted to avoid more waste and harmful influence on the natural environment in mine field in order to improve the usage rate of water. the technique with a process going on like delamination-dumping-making land- reclamation is the best one to be used in open mine field. Mining along with filling the holes with waste rock and tailing is a better way to be adopted. Some better exploitation techniques good at keep the earth in its former level should be promoted, just as exploitations in ribbon structure or in lamination. The waste water coming out of mining process must be recycled in term of some proper formulation, management and exploitation. Experts should try their best to both increase the times of using and make full use of resources in designing a mine factory. Some measures as planting trees should be taken to avoid soil erosion and landslide caused by open pit, waste rock factory, tailing garage and so on. What kind of plants should be chosen carefully is an important decision asking overall consideration. Furthermore, genetic engineering should be involved in reusing waste land, and the best design on landscape need to be adopted.
5 Filling mining research

To sum up, mining along with filling by solid waste in mines is the main force of the so-called green mining. In terms of the present techniques, tailings and waste rock are still the main sources to be used to fill the holes of mines. Tailings appear more in the two. Therefore, Tailings dry emissions technology needs to work in mines, at the same time, some researches on it need to be conducted, including compressing ettle with low payment in the industrial filling techniques, transmission techniques in big scale in the well, the indispensable filling techniques in high speed, and the way of controlling pollution. The aims consist of three aspects. The first one is picking up tailings without entering the area. Sequential filling with gob area is the second one. The last one we need to achieve is studying the computerized techniques. It is a need to advocate the present skill with two processes happening at the same time that are mining and filling with tailing consolidation(Bartos P. J.M. et al.,2005).

Waste rock is a main waste in mines with great quantity, but the present technology is not the best way to prevent pollution(Sven M. et al.,2005). In many mines, the advanced technology is the used fully, they take the normal waste rock as the material of filling and cement as consolidation. Besides, the cooperation have to be done in proper places, as a result, less waste rock is used in the few mines. However, in some mines as Jinchuan, waste rock is still not be used fully with cut-and-fill system. So it is the high time to do study on the new mode of filling in order to reduce the cost and pollution.

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NUMERICAL ANALYSIS AND LABORATORY EXPERIMENT ON THE FAILURE MODE OF STAGE BACKFILL STOPE DUE TO MINING DISTURBANCE

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In order to study the failure rules and mechanism of backfill stope under extraction effect systematically, based on engineering background of Hemushan iron mine, the stress state, the displacement deformation and the plastic zone changes characteristic of backfill stope during different excavation processes are simulated. The results show that the maximum excavation effect domain happens around the surrounding rock and panel pillars. The vertical displacement of the room roof increases during excavation processing. The ore-body adjacent to the mined out area leads tensile failure trends. The tensile and shear zone dominates the room roof rock, and the support pillar is under shear significantly. The failure mode of SBF stope is obtained, and the results are consistent with the numerical simulation. According to the engineering background at Hemushan Iron mine, the rock stress behaviour and roadway failure characteristics are taken and summarized, and the deformation monitoring sections of roadway are measured. The site measured data and the feature of roadway further verified the numerical and experimental results.

1 Introduction

Stage Backfill (SBF) is a mass mining method based upon the utilisation of gravity flow of blasted ore. The method functions on the principle the ore is fragmented at each sublevel, while ore is massively extracted at bottom locations and mining sequence of upper-level is prior to the lower level [1]. Current SBF method geometries (Fig.1) consist of a series of sublevels created at interval between 10 and 15 m, beginning at the top of the ore-body and working downward. A number of parallel drives are excavated on each sublevel, with drives being symmetrical arrangement between sublevels. From each sublevel drives, vertical or near vertical blast hole fans are drilled upward to the symmetrical drives at upper sublevel.
In the past few decades, with the promotion of concerns on environment consciousness and resource utilization, the introduction of a variety of advanced trackless facilities and the establishment of massive scale underground metal mines which annual output are usually multmillions tons, the SBF mining method has been becoming the primary choice for mine designers and engineers. It has the advantages of a relatively high production efficiency and intense resource recovery, but because of the straight hanging wall and the stresses induced by blasting, the stability of SBF stope is considerably severe, usually resulting in breakdown of drives adjacent to it, which have exerted a strong influence on the safety of mining staff and facilities. Thus the stability of surrounding excavated void is the crucial for the application of this SBF method, and the behaviour of surrounding rocks is complex during different processes. There are many factors influencing the stability of stope, such as the parameters related to geometric design consideration (sublevel height, crosscut spacing, drive geometry), draw control practices, and rock masses properties (friction angle, density, cohesion). A major factor influencing the stope stability is the excavation disturbance during mining ore [2, 3, 4, 5]. Despite lots of literatures demonstrated the different ways of analyzing the stability of voids by experiment results, theoretical calculation, Rick evaluation theory [6, 7, 8, 9] and Micro-seismic monitoring technique [10], which are totally mined out, the mechanics of SBF stope is not well understood during excavation process.

Based on these early results, this paper has analysed the failure mechanism of SBF during the mining process with FLAC3D simulation software from macroscopic perspective in section 3. To further understand the failure mode of void, the similar material experiment model was utilised to understand the pattern of the micro fissure behaviour on hanging wall at different stresses conditions in section 4. Finally, field monitoring dots of drives deformation and failure mode located at different sublevel at Hemushan Iron mine with SBF method are conducted to assess the stability of stope in section 5.

**Fig.1 Geometric parameters of SBF method at Hemushan Iron mine**

2 Engineering data

In this section we briefly introduced the geometric parameters of SBF stope and the geological condition at Hemushan Iron mine. The Stage Backfill method is comprised of 3 sublevels, -162.5m, -175m and -187m, irrespectively, and couples of rooms and pillars with both width of 12.5m, which are divided into three panel distict (Fig.1). Crosscut drives are 3.2m wide by 3.0m high, and spaced to 12m centres. The geological condition is complex in nature. The deposit is structurally controlled and mainly hosted at diorite and limestone with a high degree of jointed and fractured. The most component of the ore-body seriously powdered and clayed is iron ore, and the powder ore zone and block ore zone is mixed with each other. Therefore, the stability of SBF stopes is unclear and poor,
resulting in upgrading the difficult extent of mining the ore. Based on experimental results, the approximately material parameters are obtained, as it is shown in table 1. A full scale of SBF stopes model is analyzed with FLAC3D in this study (Fig.2), according to finite element procedure. To fully know the mining influence on the top rock and wall, many stress and deformation monitoring points are pre-designed in this simulation model. It is anticipated that the proposed method is able to represent the total change process of stope at different excavation period in the following sections.

Table 1 Parameter of rock mechanics

<table>
<thead>
<tr>
<th>Name</th>
<th>Bulk modulus/×GPa</th>
<th>Possion’s ratio /μ</th>
<th>Uniaxial compress strength /MPa</th>
<th>Tensile strength /MPa</th>
<th>Friction angle (/°)</th>
<th>Density /(kN/m3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diorite</td>
<td>3.3~4.9</td>
<td>0.13~0.28</td>
<td>17.7~39.0</td>
<td>1.46</td>
<td>26~59</td>
<td>27</td>
</tr>
<tr>
<td>Ore</td>
<td>10.8~13.9</td>
<td>0.17~0.34</td>
<td>50.2~81.1</td>
<td>2.0</td>
<td>34~37</td>
<td>32</td>
</tr>
<tr>
<td>Limestone</td>
<td>12.1~24.3</td>
<td>0.14~0.27</td>
<td>83.~127.3</td>
<td>2.5</td>
<td>42~48</td>
<td>29</td>
</tr>
</tbody>
</table>

Fig.2 A full scale layout of SBF model

3 Analysis on the stability of SBF stope

The SBF method was adopted for the deposit, with mining beginning at the -162.5m level (approximately 200 m below surface), and advancing sequentially to depth. The upper production operation affects the stability of lower drives, thus, often lead over the lower production horizontal. Mining disturbance is a complex process with the development of production levels advancing, leading to the change of stresses and deformation of surrounding rock, especially to the hanging wall of pillars between panels. In summary, controlling and assuring the safety of staff is the key point, the stress and deformation change law induced by mining out ore is summarized with FLAC3D software in the following sections.

3.1 displacement change analysis

In this section, a detailed analysis of deformation change is shown. During the totally mining, the disturbance by excavation process is repeated, and the effect extent is different from those previous each time. Basically obeying the draw sequence that the lower horizontal is hysteretic to the upper level by 6 to 8m, the initial blast site begins at the slot drives, then sequentially advancing to the crosscut drive by blast holes fans with 2.0m burden. Taking the top rock of 2nd room as research object, the disturbance effect process is analyzed by being layout a series of stress and displacement monitoring.
points at different sites (Fig.2), NO.1 to No.4 is set at the hanging wall of pillar between 1st panel and 2nd panel, No.5 to No.7 at the top of 2nd room with distanced by 3m, 15m and 30m away from it, irrespectively. The monitoring results show that the disturbance on the displacement of room top rock is subtle and the displacement values at No.6 and No.7 is small at initial period, compared to the values when the recovery work face advances 18m away from slot cut (Fig.3), and it also is demonstrated that the disturbance domain is far from the panel pillars. From the changes of displacement values, we concluded that the movement of top rock related to the exposure areas, the longer distance of excavation along recovery direction is, the maximum displacements is larger, approximately reaching 50mm. the horizontal deformation of pillars is changed by the process approach, the change at the middle of pillar is largest, approximately about 79mm, at which it is vulnerable to break and stability is weakest.

3.2A analysis on stress by disturbance

A major factor influencing the stability of the SBF stope at stress balance state initially is the concentrated stresses induced by production excavation, usually resulting in the breakdown of openings and drives, and the excavation extent is another influencing one. The main stress distribution of SBF stope at different stages and differences between top rock and hanging wall are shown at Fig.4. The top is tensile with semi-elliptical ring, whose extent increases with the distance from work face to slot cut, and the stress at hanging wall adjacent to mined out area is released, the effect zone gradually extend to the panel pillars, which also makes it tensile at final process, as shown at Fig.4(a)-(d). The most vulnerable domain mainly focuses on the top rock and the openings intersected at panel districts, so the different support schemes should be taken into consideration, depending on the stability and excavation disturbance.
3.3 Analysis plastic zone by disturbance

The top of room is under plastic tensile strain, upper zone under shear strain, the area is increased rapidly as the work face advances along the mining direction, and the state of plastic at hanging wall and panel pillar is similar, as is illustrated at Fig.5. The concentrated stress zone is about 6m away from it, which depends on the extent of mining activity. Before the workface extend the 4th step (approximately 14m away from middle slot cut), the plastic zone is tiny, and the SBF stope is considerably stable. Then, the room top plastic zone is through the panel pillars, leading to expanding the shear strain territory, as shown Fig.5 (b)-(c). At last, the panel pillars are mainly in shear strain (Fig.5 (d)). The distribution of plastic shear strain in figs.5 and stress strain in figs.4 are very similar, which is due to the fact that excavation disturbance affect the domain similarly. Therefore, from both of the stress strain distribution and plastic strain distribution, we conclude that the simulation model can illustrate the process of stability of SBF stope apparently, and the distribution law of stress strain and plastic strain are summarized as follows: the excavation activity is the main factor inducing the rock stress re-distribution which is originally is balanced, and the change is ranged from the location to
another. Generally, the top rock of mined out area is tensile failure and the hanging wall is shear deformation. To a extent, the shear failure is the primary cause to the sidewall of the drive adjacent to mined out area, which provide a good information the support measure should be taken to prevent the stability of rock around the void timely and appropriately.

Fig5. The plastic zone distribution at different steps: (a) plastic strain distribution section at 2nd step; (b) plastic strain distribution section at 4th step; (c) plastic strain distribution section at 6th step; (d) the final plastic strain distribution section

4 The SBF stope failure mode

The characteristic of deformation is fully described previously, to further understand the SBF stope failure mode, in this section, the failure mode of SBF stope is presented. The simulation material model was built to illustrate the deformation and failure process of stope during mining disturbance with blast holes fans, which is a good and convenient way to study the failure feature of massive extraction process. The sand, quicklime and gypsum powder are originally component, the simulated model is built by geometry ratio by 1 to 100. The Dial indicator, stress gauge and the transit survey are resorted to recording the horizontal movement of panel pillar, stress change and surface displacement, irrespectively. The initial rock stress is generated by liquefied compress system set on the top of the test model until it failed. The records are collected step by step when the system working. All of the recording instruments are arranged as shown in Fig.6.
It is obviously that the failure is happened to the top rock and panel pillar, when the pressure is turn on. At the beginning of test, the tiny cracks appeared ant then a small area rock is separated off, the shape of the failure belt is similar to a arching. As can be seen in Fig.7, the panel pillar occur shear movement when the pressure is equal to 10MPa, and the cracks propagate along the pillars between two panel, as illustrated in Fig.7a, at the same, the horizontal displacement get to 45mm. These cracks are propagated instantly as the additional pressure is increased. Finally, one sidewall of drive begins to move award the mined out area (Fig.7b), the deformation of drives at different levels is apparent. The top stress provided by liquefied system reaches 45MPa, the stop failed, and the pillar move to voids along an angle fracture section surface about approximately 60°, the Fig.8 present a series of displacement values at different pressure.

A summary of experiment results are presented, the top rock of room is caved freely due to tensile effect, the domain depends on the surrounding pressure, and the stope will keep stable when the stress is re-balanced. The shape of caving area is like a elliptical arching. The shear failure mode of pillar is the dominant form.

Fig.6 The similar material test model

![Fig.6 The similar material test model](image)

Fig.7 The failure mode of panel pillar.(a):Loading is 20MPa;(b):Loading is 35MPa;(c)Before failure, namely loading is 45MPa.

![Fig.7 The failure mode of panel pillar](image)
5 On-site monitoring and result analysis

The SBF method has been implemented since 2009 at Hemushan Iron mine, located 15 km to the Ma-anshan city, Ma Steel group, An-Hui province, China, which is subdivided to four panels, and the panel spacing is 50 m. There is always a approximately 6 or 8m wide pillar set aside for the roadway connected with the drives at each level, thus, the importance of keeping the roadway stability is particularly obvious before the orebody is completely extracted. To fully get the deformation records and the failure mode of drives, the field investigation procedure of openings breakage were taken, and the layout of the monitoring points were fixed at the both sides of the drives wall on each level [11].

There are 12 monitoring section fixed up at -162.5m, -175m and -187m level, irrespectively, and the type JASS30-15 digital convergence gauge is used to measure the drive surface deformation, as shown in Fig.1(b). As can be seen at table 2, the drives deformation changed differently due to the position and excavation disturbance, there is almost no change to the roadway at -187m level except that the number -187m-1 section converged by 5mm. Compared with the another two levels, the drives at -175m level are more unstable, the maximum convergence value get to 11mm. The expansion of 13mm happened to the number -162.5m-2# monitoring section, and the reason is that the side wall of the roadway has moved toward to the mined out area caused by the mining activity on the 15th, march as illustrated at Fig.9.

<table>
<thead>
<tr>
<th>Layout point</th>
<th>2010-4-9</th>
<th>2010-5-12</th>
<th>2010-6-28</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>-187-1 section</td>
<td>3.1154</td>
<td>3.1126</td>
<td>3.1105</td>
<td>Convergence 5mm</td>
</tr>
<tr>
<td>-187-3 section</td>
<td>2.6799</td>
<td>2.6794</td>
<td>2.6794</td>
<td>No change</td>
</tr>
<tr>
<td>-175-1 section</td>
<td>2.5259</td>
<td>2.5268</td>
<td>2.2563</td>
<td>No change</td>
</tr>
<tr>
<td>-175-2 section</td>
<td>3.2250</td>
<td>3.2160</td>
<td>3.2141</td>
<td>Convergence 11mm</td>
</tr>
<tr>
<td>-175-3 section</td>
<td>2.4087</td>
<td>2.4031</td>
<td>2.4022</td>
<td>Convergence 6.5mm</td>
</tr>
<tr>
<td>-162.5-2 section</td>
<td>2.8070</td>
<td>2.8012</td>
<td>2.8034</td>
<td>Convergence 3.6mm</td>
</tr>
<tr>
<td>-162.5-3 section</td>
<td>2.5640</td>
<td>2.5645</td>
<td>2.5643</td>
<td>No change</td>
</tr>
<tr>
<td>-187-2# section</td>
<td>3.0250</td>
<td>3.0265</td>
<td>3.0250</td>
<td>No change</td>
</tr>
<tr>
<td>-162.5-2# section</td>
<td>2.6279</td>
<td>2.6277</td>
<td>2.6436</td>
<td>Expand 13.0mm</td>
</tr>
</tbody>
</table>
6 Conclusions and Future Work

In this paper we have analysed the failure mode of the roadway caused by excavation process disturbance. Specifically, we have focused on the drive deformation characteristics on site. The work described is focused on investigating the rock stress change of surrounding rock resulted from the mining, and some conclusions are summarized as follows:

(1) The stability of SBF stope is analysed through FLAC3D numerical simulation method during mining process, and we know that the vulnerable area usually happened to the top rock and panel district pillars.

(2) The reason resulted in top rock failure is tensile stress strain, and panel district pillar is shear stress strain. The plastic strain increased with the proximity of the exaction distance, leading to the poor stability, and probability of breakdown is going to occur easily.

(3) The failure mode of SBF stope is obtained, and the results are consistent with the numerical simulation.

(4) According to the engineering background at Hemushan Iron mine, the rock stress behaviour and roadway failure characteristics are taken and summarized, and the deformation monitoring sections of roadway are measured. The site measured data further prove the numerical and experimental results.

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RANDOM SOLID-FLOW-HEAT-GAS COUPLED MODEL OF CONTINUOUS ROCK MEDIUM

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We took the continuous rock medium account, and used probability and mathematics statistics method, the mechanics of elasticity, thermodynamics theory, the fluid theory and heat conduction theories and research methods of structural mechanics, furthermore. Based on the difference of the rock’s micro-structure and random distribution, we treated physical mechanics parameters as the random medium and built 3-D random non-homogeneous solid-flow-heat-gas coupled mathematics model of continuous rock medium and its FEM. This paper analyzes features of random non-homogeneous solid-flow-heat-gas coupled mathematics mode deeply. This model is the development of the classical solid-flow coupled mathematics model, solid-heat coupled mathematics model and the solid-flow-heat coupled mathematics model, and it offers the theoretical foundation and methods to study the rock deformation.

1 Introduction

Coupled mathematics model of Continuous rock medium and the solution to its numerical value have been one of important topics in the rock and soil mechanics filed. Papers [1, 2] have studied the inhomogeneous material mechanics model and its related calculation method. And by using non-uniform finite element model, it simulates unstable break phenomenon of rocks. Papers [3, 4] introduce solid-flow-heat coupled mathematics model of matrix-fractured rock medium in detail, and it shows the 3-D numerical simulation to the HDR geothermal development. Currently coupled mathematics model of the 3-D inhomogeneous material is just on the beginning stage. Papers [5-8] has studied random solid-flow-heat coupled model of the rock medium, thermal elastic mechanics model of the random medium’s analytical solution to the spherical coordinate problem and the plane axial symmetry problem, the solid-heat coupled model of the rock medium’s analytical solution to the plane axial symmetry problem. Considering about the influence of the gas to inhomogeneous rock medium, this paper takes some study methods including probability theory and mathematics statistics, elastic mechanics, thermodynamic theory, heat transferring theory and structure mechanics to build random solid-flow-heat-gas coupled model of Continuous rock medium.

2 Basic hypothesis and the physical mechanics foundation

To study conveniently, the content below is regarded as the basic hypothesis and the physical mechanics foundation.

1) The rock is made up of random homogeneous particles in microcosmic, however, as for the micro-unit which is simplified, it also constitutes many mineral particles. On macroscopic, the
micro-unit is the homogeneous isotropic elastic medium.

2) Micro-unit’s characteristic parameters of the physical mechanics are the statistical characteristics of a numeral number of mineral particles.

3) Heat transfer in the rock is only existed by conduction, regardless of the heat transfer via the convection and radiation.

4) These Micro-units’ dimensions are very small, and they do not have the macroscopic statistical characteristics, so the unit characteristics expressed by the micro-unit experiences random heterogeneity as the whole.

5) As the micro-unit’s heterogeneity, and at the same time it reflects the elastic constants, the coefficient of heat conduction, the coefficient of thermal expansion, the coefficient of heat capacity, internal friction angle, the cohesion, the shear and compressive strength and other characteristics which also have the random heterogeneity. It is hypothesis that the randomness of these parameters and heterogeneity are unified.

6) Based on the above hypothesis of the random heterogeneity, virtually the macroscopical performance is the anisotropy, strictly speaking, its stress balance equation and heat conduction equation should be the anisotropic equation. But as this kind of anisotropy is random and hard to express, the paper only consider about the overall heterogeneity, and it do not put the anisotropy created aside.

7) The rock mass medium liquid seepage rule obeys Darcy’s law:

\[ q_i = K_{ii} p_j \quad i = (1, 2, 3) \]

In this equation, \( K_{ii} = K_i(\theta, p) \) is the Coefficient of permeability, \( K_{ii} \) is the function of the volume stress \( \theta \) and the pore pressure \( p \).

8) Under the action of high temperature and high pressure, the liquid density is no longer a constant, but it is a pressure and temperature function. The liquid density is:

\[ \rho_l = \rho_l(T_l, p_l) \]

In the equation, \( \rho_l \) is the liquid density (g/cm\(^3\)); \( T_l \) is the liquid temperature (°C); \( p_l \) is the liquid pressure, the unit is the absolute atmospheric pressure.

9) It is hypothesis that the flow of the liquid belongs to the forced convection, the flow velocity is not influenced by the density and the viscosity, namely, the flow velocity is not influenced by the temperature field.

10) The volume deformation of the saturated porous medium constitutes two parts which are the rock solid skeleton deformation and the pore deformation. \( \alpha_b = (1-n)\alpha_s + n\alpha_p \) It is hypothesis that \((1-n)\alpha_s \leq n\alpha_p\), so the volume deformation of the saturated porous medium is equal to the
pore deformation. In the equation, $\alpha_b$, $\alpha_s$, $\alpha_p$ is the overall deformation, the rock solid skeleton deformation, the pore deformation, respectively.

11) The gas content in the rock medium obeys Lang's formula:

$$ C = n\rho + ab\rho/(1 + b\rho) $$

12) The gas can be treated as the ideal gas, the seepage flow can be disposed by the isothermal process. The gas state equation is:

$$ \rho = p / RT $$

13) The rock medium’s pore and its fracture are saturated by the single phase water and gas.

14) The gas in the rock obeys the linear Darcy law:

$$ q_q = K_{qj}p_i \quad i = (1,2,3) $$

In the equation, $K_{qj}$ is the permeability coefficient, $K_q$ is the function of the volume stress $(\theta)$ and the pore pressure $(p)$.

3  The 3-D random solid-flow-heat-gas coupled mathematics model of Continuous rock medium

3.1. The solid deformation control equation of the rock medium

Based on the hypothesis 1) and the basic theory of the elastic mechanics, the stress balance equation of the rock mass is:

$$ \sigma_{ij,j} + F_i = 0 $$(1)

The stress balance equation of the rock mass is:

$$ (\lambda_r + G_r) \frac{\partial \varepsilon_r}{\partial x_i} + G_r \nabla^2 u_i + \frac{\partial \lambda_r}{\partial x_i} \varepsilon_r + \sum_{j=1}^3 \frac{\partial G_r}{\partial x_j} \frac{\partial u_j}{\partial x_i} + \sum_{j=1}^3 \frac{\partial G_r}{\partial x_i} \frac{\partial u_j}{\partial x_j} + F_i = \left( i = 1,2,3 \right) $$(2)

Among them, $\lambda_r, G_r$ are Lame constant; $u$ is the displacement; $F$ is the external force; $\beta_r$ is the thermal expansion coefficient of the rock block; $\varepsilon_r$ is the volumetric strain; $T_r$ is the temperature of the rock block; $K_r$ is the volume deformation modulus of the rock block. $\lambda_r, G_r, \beta_r$ and $K_r$ are non-homogeneous random variables which obey a random distribution.
Namely, \( \lambda_i = \lambda_i(x_1, x_2, x_3, t) \); \( G_i = G_i(x_1, x_2, x_3, t) \); \( \beta_i = \beta_i(x_1, x_2, x_3, t) \);
\( K_i = K_i(x_1, x_2, x_3, t) \).

3.2. The liquid seepage equation of the rock medium

According to the hypothesis 7)-9), the conservation of mass law of the liquid in the rock is:

\[
div (\rho_i q_i) = \frac{\partial (\rho_i n)}{\partial t}
\]

(3)

Based on the hypothesis 10), \( \frac{\partial \varepsilon}{\partial t} = \frac{\partial n}{\partial t} \), and as the micro compressibility of the water which can be expressed by \( \frac{\partial \rho_i}{\partial t} = \beta_i \rho_i \frac{\partial n}{\partial t} \), then we obtain

\[
div (\rho_i q_i) = -\beta_i \rho_i n \frac{\partial p_i}{\partial t} - \rho_i \frac{\partial \varepsilon}{\partial t} - W_i
\]

(4)

Its equation is :

\[
\frac{\partial}{\partial x_1} \left( \kappa_i \frac{\partial p_i}{\partial x_1} \right) + \frac{\partial}{\partial x_2} \left( \kappa_i \frac{\partial p_i}{\partial x_2} \right) + \frac{\partial}{\partial x_3} \left( \kappa_i \frac{\partial p_i}{\partial x_3} \right) = \beta_i n \frac{\partial p_i}{\partial t} + \frac{\partial \varepsilon}{\partial t} + W_i
\]

(5)

Then we get:

\[
\kappa_i \nabla^2 p_i + \sum_{i=1}^3 \frac{\partial \kappa_i}{\partial x_i} \frac{\partial p_i}{\partial x_i} = \beta_i n \frac{\partial p_i}{\partial t} + \frac{\partial \varepsilon}{\partial t} + W_i
\]

(6)

Rock medium’s thermal elastic constitutive relation is

\[
\sigma_{ij} = \lambda \delta_{ij} \varepsilon_{kk} + \frac{E}{1 + \nu} \varepsilon_{ij} - \frac{E}{1 - 2\nu} \beta \Delta T \delta_{ij}
\]

(7)

Among them, \( p_i \) is the pressure of the liquid; \( t \) is the time; \( \kappa_i \) is the permeability coefficient of the liquid; \( \rho_i \) is the density of the liquid; \( n \) is the porosity; \( W_i \) is the source sink; \( q_i \) is the seepage velocity; \( \beta_i \) is the compressibility coefficient of the liquid; \( \lambda \) is the Lame constant; \( E \) is the elasticity modulus; \( \nu \) is the Poisson’s ratio; \( \Delta T \) is the temperature increment of the rock;
\( \delta_{ij} \) is the Kronecker symbol; \( \varepsilon \) is the volume strain of the rock. \( \kappa_i \) is the non-homogeneous random variable which obeys a random distribution, namely, \( \kappa_i = \kappa_i(x_1, x_2, x_3, t) \)

3.3. The temperature field control equation of the rock medium

As for the rock heat conduction problem, based on the hypothesis (3) and (5), the energy conservation equation is:

\[
\frac{\partial}{\partial t}\left( \kappa_i \frac{\partial T}{\partial x_i} \right) + \sum_{i=1}^{3} \frac{\partial}{\partial x_i}\left( \kappa_i \frac{\partial T}{\partial x_i} \right) + Q_0 = \frac{\partial (\rho c T)}{\partial t}
\]

(8)

The unsteady heat conduction equation of the heterogeneous stroma rock block is:

\[
\kappa_i \nabla^2 T_i + \sum_{i=1}^{3} \frac{\partial \kappa_i}{\partial x_i} \frac{\partial T_i}{\partial x_i} + Q_0 = \rho c T_i \frac{\partial T_i}{\partial t} + \rho T_i \frac{\partial c_r}{\partial t} + T_i c_r \frac{\partial \rho_r}{\partial t}
\]

(9)

Among them, \( T_i \) is the rock’s temperature; \( t \) is the time; \( \kappa_i \) is the heat conduction coefficient of the rock mass; \( c_r \) the specific heat capacity of the rock block; \( \rho_r \) is the rock density; \( Q_0 \) is the heat source sink; \( \kappa_i, c_r \) and \( \rho_r \) are non-homogeneous random variables which obey a random distribution, namely, \( \kappa_i = \kappa_i(x_1, x_2, x_3, t) \); \( c_r = c_r(x_1, x_2, x_3, t) \); \( \rho_r = \rho_r(x_1, x_2, x_3, t) \).

3.4. The gas seepage equation of the rock medium

Based on the conservation of gas mass law:

\[
\text{div}(\rho q) = \frac{\partial C}{\partial t}
\]

Its equation is:

\[
\frac{\partial}{\partial x_1} \left( \kappa_q \frac{\partial p_q^2}{\partial x_1} \right) + \frac{\partial}{\partial x_2} \left( \kappa_q \frac{\partial p_q^2}{\partial x_2} \right) + \frac{\partial}{\partial x_3} \left( \kappa_q \frac{\partial p_q^2}{\partial x_3} \right) = \left[ \frac{n}{p_q} + \frac{ab}{p_q \left( 1 + bp_q \right)^2} \right] \frac{\partial p_q^2}{\partial t} + 2p_q \frac{\partial n}{\partial t}
\]

(10)

The gas flow equation of the heterogeneous stroma rock block is:

\[
\kappa_q \nabla^2 p_q^2 + \sum_{i=1}^{3} \frac{\partial \kappa_q}{\partial x_i} \frac{\partial p_q^2}{\partial x_i} = \left[ \frac{n}{p_q} + \frac{ab}{p_q \left( 1 + bp_q \right)^2} \right] \frac{\partial p_q^2}{\partial t} + 2p_q \frac{\partial n}{\partial t}
\]

(11)

Among them, \( C \) is the gas content; \( p_q \) is the gas pressure; \( t \) is the time; \( \kappa_q \) is the gas permeability coefficient; \( a, b \) is the adsorption coefficient; \( \rho_q \) is the gas density; \( n \) is the porosity;
\( \kappa_q \) is the non-homogeneous random variable which obeys a random distribution, namely,

\[ \kappa_q = \kappa_q(x_1, x_2, x_3, t) \]

### 3.5 Random solid-flow-heat-gas coupled mathematics model

The stress balance equation of the heterogeneity rock mass is:

\[
(\lambda_r + G_r) \frac{\partial \varepsilon_r}{\partial x_r} + G_r \nabla^2 u_r + \frac{\partial G_r}{\partial x_i} \varepsilon_r + \sum_{i=1}^{3} \frac{\partial G_r}{\partial x_j} \frac{\partial u_r}{\partial x_j} + \sum_{i=1}^{3} \frac{\partial G_r}{\partial x_i} \frac{\partial u_r}{\partial x_i} + F_i = 0 \quad (i = 1, 2, 3)
\]

The thermal elastic constitutive relation of the rock medium is:

\[
\sigma'_i = \lambda \varepsilon_{kk} + \frac{E}{1 + \nu} \varepsilon_{ij} - \frac{E}{1 - 2
u} \beta \Delta T \varepsilon_{ij}
\]

Liquid seepage control equation of the heterogeneity rock mass is:

\[
\kappa_i \nabla^2 p_i + \sum_{i=1}^{3} \frac{\partial \kappa_i}{\partial x_i} \frac{\partial p_i}{\partial x_i} = \beta_r \rho_r \frac{\partial p_i}{\partial t} + \frac{\partial p_i}{\partial t} + W_i
\]

Unsteady heat conduction equation of the heterogeneity rock mass is:

\[
\kappa_q \nabla^2 T_r + \sum_{i=1}^{3} \frac{\partial \kappa_q}{\partial x_i} \frac{\partial T_r}{\partial x_i} + Q_0 = \rho_r c_r \frac{\partial T_r}{\partial t} + \rho_r c_r \frac{\partial c_r}{\partial t} + T_r c_r \frac{\partial p_r}{\partial t}
\]

The gas seepage equation of the heterogeneity rock mass is:

\[
\kappa_q \nabla^2 p_q^2 + \sum_{i=1}^{3} \frac{\partial \kappa_q}{\partial x_i} \frac{\partial p_q^2}{\partial x_i} = \left[ \frac{n}{p_q} + \frac{ab}{p_q(1 + bp_q)^2} \right] \frac{\partial p_q^2}{\partial t} + 2 \rho_q \frac{\partial n}{\partial t}
\]

Equations of (12) and (16) constitute random medium solid-flow-heat-gas coupled mathematics model.

Compared with the homogeneous solid-flow-heat coupled mathematics model, the proposed random medium solid-flow-heat-gas coupled mathematics model has the following characteristics:

1. It augments the gas seepage equation of the heterogeneous rock mass, which exerts great influence on the rock deformation.
2. The rock balance equation augments 5 items, namely, the deformation item \( 3 K T_r \frac{\partial \beta_r}{\partial x_i} \) caused by the coefficient of thermal expansion \( \beta_r \) gradient, the deformation item \( 3 \beta_r T_r \frac{\partial K}{\partial x_i} \) caused by the modulus of the volume deformation \( K \) gradient, the deformation item \( \frac{\partial \lambda}{\partial x_i} \varepsilon_{ij} \)
caused by Lame constant \((\lambda)\) gradient and deformation items \(\sum_{j=1}^{3} \frac{\partial G_{i}}{\partial x_{j}} \frac{\partial u_{j}}{\partial x_{j}} + \sum_{j=1}^{3} \frac{\partial G_{i}}{\partial x_{j}} \frac{\partial u_{j}}{\partial x_{j}}\) and 
\[\varepsilon_{i} \frac{\partial G_{i}}{\partial x_{i}}\]
caused by Lame constant \((G_{r})\) gradient.

Previous thermal elastic mechanics equation only considered about the deformation item caused by temperature gradient, while the above 5 items were not taken into account. In the study of the rock thermal fracturing, these 5 items account for a large proportion, specially the deformation item \((3KT_{r} \frac{\partial \beta_{r}}{\partial x_{i}})\) caused by the coefficient of thermal expansion \((\beta_{r})\) gradient exerts great influence on the rock deformation.

(3) The heterogeneous heat conduction equation of the rock medium augments 3 items, namely,
the deformation item \((\rho_{r}T_{r} \frac{\partial c_{r}}{\partial t})\) caused by the specific heat capacity \((c_{r})\) gradient, the deformation item \((\sum_{i=1}^{3} \frac{\partial k_{r}}{\partial x_{i}} \frac{\partial T_{r}}{\partial x_{i}})\) caused by the heat conduction coefficient \((k_{r})\) gradient, specially the deformation item \((\sum_{i=1}^{3} \frac{\partial k_{r}}{\partial x_{i}} \frac{\partial T_{r}}{\partial x_{i}})\) caused by the heat conduction coefficient \((k_{r})\) gradient exerts great influence upon the rock deformation.

(4) The liquid density is not disposed as the constant. It is the function of the water pressure and the water temperature, namely, \(\rho_{w} = \rho_{w}(T_{w}, p)\)

(5) As the heterogeneity of the micro-unit, the elastic constant reflected also has the heterogeneity, in this sense, equation (12)—(16) are random solid-flow-heat-gas coupled model of continuous rock medium In general sense

4 The random medium solid-flow-heat-gas coupled mathematics model of the finite-element analysis.

If study problems’ basic equations, their definite solution conditions and their related condition are given, the whole definite solution problem of random medium solid-flow-heat-gas coupled will be obtained. It is hard to use analytic method to solve such difficult problem. The finite element numerical method of the rock medium multi-field coupled mathematics model has been introduced in many articles in detail. The random finite element numerical method treats physical mechanics parameter as the random variable in the above numerical method, and then we can value assignment in the finite element.
5 Conclusion

(1) The paper comes up with 3-D random non-homogeneous the solid-flow-heat-gas coupled mathematics model of Continuous rock medium and its calculation method. These equations and methods establish the basic of the rock deformation study.

(2) This mathematics module is the extension and the development of the solid-flow-heat-gas coupled mathematics model, which is more practical.

(3) We can draw a conclusion that random non-homogeneous characteristics of the rock exerts imperative influence upon the rock thermal fracture via analyzing solid-flow-heat-gas coupled model of the random medium.

(4) The gas flow equation of the heterogeneous rock medium exerts great influence on the rock deformation.

References
DIGITAL IMAGE PROCESSING AND FRACTAL DIMENSION CALCULATION OF ROCK FRACTURE IMAGES

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The basic theories and methods of digital image processing are simply introduced. Using digital image processing technology, the images of rock fractures provided by a scanning electron microscope (SEM) are gray-scale processed, smooth filtered, segmented by threshold selected by Otsu’s method, edge detected, then the binary (black and white) bitmaps of those images are obtained. After that their fractal features are extracted and their fractal dimension values are calculated using box dimension method. The fractal geometry models of the different modes of rock fractures are also established. The fractal dimension values of those models are evaluated and compared with the fractal dimension calculation results from digital image processing. At last the relationship between the mode of rock fracture and the fractal dimension of its image is illustrated.

1. Introduction

Digital image processing can handle images such as remote sensing photos, SEM photos, special images obtained through CT technology, all of which contains a lot of physical information. Since a digital image is sampled at a certain resolution, it doesn't have the continuity which a geometric figure has, and its resolution is usually lower than an optical image's. Therefore, the methods of fractal dimension calculation based on digital image processing also have some characteristics.
Previous researches on digital images' fractal dimension calculation [1, 2, 3, 4, 5] and on the fractal geometry models of fractures of rock and other kinds of solid material [6, 7, 8, 9, 10, 11, 12] have been done. In this article we use a program to apply digital image processing technology to the images of rock fractures captured by SEM. The images are gray-scale processed, low-pass filtered, segmented by threshold selected by Otsu's method, edge detected, and then their fractal features are extracted to calculate their fractal dimension values using box dimension method. The theoretical bases of all of those processes above are provided. At last we compare the rock fractures’ fractal dimension calculation results with the ones obtained by fractal geometry modeling and analysis, illustrating the relationship between the mode of rock fracture and the fractal dimension of its image.

2. Primary Steps of Digital Image Pre-processing

2.1 Gray-scale Processing

Generally, the digital images we can obtain in everyday life are mostly colored and in R'G'B' (red, green and blue components) format. However, the ones used in digital image processing are mostly gray-scale, and the extra color information hardens the image processing. Therefore, an algorithm which “throws away” the color information in the input color image and converts it into a gray-scale one is needed.

We use the following evaluation

\[ Y' = K_r R' + K_g G' + K_b B' \]

To generate the gray scale \( Y' \) (luma, not luminance [13]) of a pixel in the output image from the \( R'G'B' \) values of the corresponding pixel in the input image, in which

\[ K_r = 0.299, K_b = 0.114, K_g = 1 - K_r - K_b = 0.587 \]

are conversion factors provided by ITU-R (International Telecommunication Union Radiocommunication Sector) in its BT.601 Recommendation.

2.2 Image Filtering

Image filtering is primarily for removing the useless noise in an image as much as possible, but it should also protect or enhance the contents in which we're interested in this image.

In a digital image, the noise is usually concentrated in the high-frequency part, the frequency of which is higher than the one of the interesting content, such as the overview or the edges among segments. Thus, we can easily find that after using fast Fourier transform (FFT) on an image, a low-pass filtering process in the frequency domain will remove the noise from this image.

Image filtering in the frequency domain means performing the following evaluations successively:

\[
\begin{align*}
F &= F(f) \\
G &= F \ast H \\
g &= F^{-1}(G)
\end{align*}
\]
in which $f$ means the input image (a two-dimensional matrix), $g$ means the output image (also a 2-D matrix), $F(f)$ and $F^{-1}(F)$ are FFT from $f$ to $F$ and its inverse respectively, and $H$ is a transfer function which means the process in the frequency domain.

We choose the ideal low-pass filtering function below

$$H(u,v) = \begin{cases} 
1 & d = \sqrt{u^2 + v^2} < d_0 \\
0 & d > d_0 
\end{cases}$$

as the transfer function $H$ in equation (1), in which $u,v$ represents the coordinate the current pixel has relative to the zero frequency position, $d = \sqrt{u^2 + v^2}$ means the distance from the current pixel to the zero frequency position, or the frequency the current pixel represents. $d_0$ is a (distance) threshold, the information with frequency higher than which will be removed by the function $H$.

Since the threshold $d_0$ might be badly chosen, the edges of a picture may be lost after low-pass filtering. Thus, the image enhancement is needed to compensate for the loss. Use the Laplacian operator

$$\nabla^2 f = \left(\frac{d^2}{dx^2} + \frac{d^2}{dy^2}\right)f$$

(2)

to calculate $\nabla^2 f$ from the input image $f$ then add it to $f$, and the image enhancement will be achieved. For discrete images, we use difference quotient instead of derivative, and the image enhancement from image $f(x,y)$ to $g(x,y)$ becomes such an equation

$$g(x, y) = f(x, y) + \nabla^2 f(x, y)$$

$$= 5f(x, y) - (f(x-1, y) + f(x+1, y) + f(x, y-1) + f(x, y+1))$$

or can be expressed in such a computing template

$$\begin{bmatrix}
0 & -1 & 0 \\
-1 & 5 & -1 \\
0 & -1 & 0
\end{bmatrix}$$

in which the place with an asterisk represents the current pixel.
2.3 Image Segmentation

The images are all made up of several segments, inside each of those segments there are some kinds of similarities while between two different segments there are not. Dividing an image to those segments according to such similarities is called image segmentation. Usually, image binarization is used to separate the "foreground" segment of an image from its "background" segment.

Single threshold method is used here to binarize the image, which needs a pre-selected gray scale threshold. Traverse the pixels in the input (gray-scale) image, set the corresponding pixel in the output image to white if input gray scale is higher than the threshold, and black if not.

Otsu's method [14] is used to select this segmentation threshold. Otsu's method, also known as maximum between-class variance method, is mainly focused on maximizing the variance \( \sigma_b^2 \) between the mean gray levels of "black" class and "white" class pixels. Although only a little more than 30 years have passed so far since N. Otsu put forward this method, research [15] is already done which considers this method as one of the "reasonably good thresholding methods".

2.4 Edge Detection

The fractal dimension calculation of a digital image is in fact the fractal dimension calculation of its edges. All the works we've done above are also to highlight the edges in the image. Next the edge detection technology will be used to extract edge features from the binary image.

Combine the following Gaussian operator

\[
\frac{1}{2\pi\sigma^2} e^{-\frac{x^2+y^2}{2\sigma^2}}
\]

with Laplacian operator (2), and the following LoG (Laplacian of Gaussian) operator

\[
H(x, y) = \frac{1}{\pi\sigma^4} \left( \frac{x^2 + y^2}{2\sigma^2} - 1 \right) e^{-\frac{x^2+y^2}{2\sigma^2}}
\]  

(3)

will be used for edge detection.

After convolution of the image \( f(x, y) \) and operator (3), if a pixel in the result matrix meets the following condition:

a) \( f(x, y) < 0 \), and \( \exists a \in \{ f(x-1, y), f(x+1, y), f(x, y-1), f(x, y+1) \} \) which meets \( a > 0 \); or

b) \( f(x, y) = 0 \), and \( \exists b \in \{ f(x-1, y)f(x+1, y), f(x, y-1)f(x, y+1) \} \), which meets \( b < 0 \)
then this pixel is an edge pixel in \( f(x, y) \). Find all the edge pixels and the edge detection result is achieved.

3. Fractal Dimension Calculation of Digital Images

3.1 Some Basic Characteristics of a Fractal

Falconer deems that if \( F \) is a fractal, then we can usually say:

a) \( F \) is extremely irregular. Far more complex than an Euclidian geometry such as a triangle or a square;

b) Exists a part of \( F \) which is similar to \( F \) itself, though perhaps the similarity is only an approximate one;

c) \( F \) has very detailed structures, from which large amount of detail can always be observed, no matter how it is zoomed;

d) \( F \) is usually generated according to some easy forms such as recursion;

e) \( F \) has some kinds of fractal dimension, which is always strictly greater than its topological dimension.

3.2 The Hausdorff Dimension of a Fractal

Suppose the diameter of a set \( U \) is

\[
|U| = \sup \{|x - y| : x, y \in U\}
\]

and call the set of sets \( \{U_i\} \) a \( \delta \)-cover of \( F \), if

\[
F \subseteq \bigcup_{i=1}^{\infty} U_i, 0 < |U_1| |U_2| |U_3| \cdots \leq \delta
\]

then we can call that

\[
H^p(F) = \lim_{\delta \to 0} \inf\left\{ \sum_{i=1}^{\infty} |U_i|^p : \{U_i\} \text{ is a } \delta - \text{cover of } F \right\}
\]

is the \( p \)-dimensional Hausdorff measure of \( F \). Generally, \( H^p(F) \) is almost always infinite or zero, but there is a value \( p \) which meets

\[
H^q(F) = \begin{cases} \infty & q < p \\ 0 & q > p \end{cases}
\]

and the value \( p \) is called the Hausdorff dimension of \( F \).
The Hausdorff dimension $p$ of a fractal is usually obtained by analysis of the fractal, and uneasy to be measured from a (drawn) figure. Thus the following concept of "box dimension" is introduced.

### 3.3 The Box Dimension of a Fractal

The box dimension is also based on the idea of covering the fractal figure with multiple small sets, however the "small sets" here can be chosen as grids, which is absolutely application-friendly.

Cover the figure $F$ with $\delta$-grids, and let $N(F,\delta)$ be the minimum amount of grids needed. If the limit

$$\lim_{\delta \to 0} \frac{\log N(F,\delta)}{-\log \delta}$$

exists, then it is called the box dimension of $F$.

### 3.4 The Fractal Dimension Measurement of Digital Images Using Box Dimension Method

In real fractal dimension measurement of digital images, what we usually get are pairs of $\log N(F,\delta)$ and $\log \delta$, then the evaluation of the limit in (4) becomes difficult. Therefore curve fitting is used for the evaluation of an approximate fractal dimension of the images.

First we should define the size of the grids. Here we choose

$$l = \lceil \log_2 (\min(h, w)) \rceil$$

Then we divide the height $h$ of the image into the following $\lceil h/l \rceil$ parts

$$1 \sim l, l + 1 \sim 2l, 2l + 1 \sim 3l, \ldots, \left(\left\lceil \frac{h}{l} \right\rceil - 1 \right)l + 1 \sim h$$

and the width $w$ into $\lceil w/l \rceil$ parts respectively. Therefore the whole image is divided into

$$\lceil h/l \rceil \times \lceil w/l \rceil$$

grids.

Next for each of those grids we find out whether there are any edge pixels in it, and $N(F,l)$, the number of grids which contain (at least one) edge pixel(s) can be counted, then $\log_2 N(F,l)$ can be evaluated.

At last we fit the point $(\log_2 l, \log_2 N(F,l))$ set with a straight line (a linear function), then the opposite of its slope is the box dimension of the image.
In summary, the following flow chart of fractal dimension calculation of digital images is used.

![Flow chart used in fractal dimension calculation of digital images](image)

**Figure 1** Flow chart used in fractal dimension calculation of digital images

### 4. Fractal Dimensions of SEM Photos of Rock Fracture

Xiao-yun Shan [16] used SEM to microscopic analyze the different kinds of rock fragments around the rupture hole wall, and study the microforms of buckled-by-impact rock fractures. Some kinds of microforms of buckled rock fractures are shown in Figure 2 to 5:

![Granite tensile fracture form](image)

**Figure 2** Granite tensile fracture form

![Diorite tensile fracture form](image)

**Figure 3** Diorite tensile fracture form

![White marble tensile fracture form](image)

**Figure 4** White marble tensile fracture form

![Diorite shear fracture form](image)

**Figure 5** Diorite shear fracture form

Xiao-yun Shan [16] put forward the fractal models of rock's intercrystalline failure, intracrystalline failure, intercrystalline coupled with intracrystalline failure, as is shown in
The fractal dimensions of the models above can be evaluated using similar dimension formula. The values are respectively:

Figure 6 (intercrystalline) (a): \[ D = \frac{2}{\ln \sqrt{3}} \approx 1.262 \]

Figure 6 (intercrystalline) (b): \[ D = \frac{4}{\ln 3} \approx 1.262 \]

Figure 7 (intracrystalline): \[ D = \frac{3}{\ln \sqrt{5}} \approx 1.365 \]

Figure 8 (coupled): \[ D = \frac{5}{\ln \sqrt{13}} \approx 1.255 \]

He-ping Xie [17] derived the equation which evaluates the critical expanding force \( G_{crit} \) when it has a fractal dimension \( D \):

\[
G_{crit} = 2r_s \left( \frac{1}{r} \right)^{(D-1)}
\]  \( (5) \)

in which \( r_s \) means the surface energy per macro measure, \( r \) means self-similar ratio.

According to equation (5), the \( G_{crit} \) values of fractures corresponding to the models above are respectively:

Figure 6 (intercrystalline) (a): \[ G_{crit} = \sqrt{3}^{0.262} \times 2r_s \approx 1.15 \times 2r_s \]

Figure 6 (intercrystalline) (b): \[ G_{crit} = 3^{0.262} \times 2r_s \approx 1.33 \times 2r_s \]
Figure 7 (intracrystalline): \[ G_{\text{crit}} = \sqrt{5}^{0.365} \times 2r_s \approx 1.34 \times 2r_s \]

Figure 8 (coupled): \[ G_{\text{crit}} = \sqrt{13}^{0.255} \times 2r_s \approx 1.39 \times 2r_s \]

When rocks are fractured in those modes above, the model analysis values of fractures' fractal dimension and the \( G_{\text{crit}} \) values of the corresponding fractures are shown in Table 1.

The higher \( G_{\text{crit}} \) value a mode of fracture has, the higher the possibility that it happens. Thus, we deem that the intercrystalline and coupled failures are more likely to happen than the intracrystalline failure. In other words, the higher the fractal dimension a fracture mode has, the more difficult it will happen.

<table>
<thead>
<tr>
<th>Fractal dimension by model analysis</th>
<th>Gerit value when grain size is 10-2cm</th>
<th>Difficulty to have this mode of fracture happen with the same kind of material</th>
</tr>
</thead>
<tbody>
<tr>
<td>Intercrystalline failure (a)</td>
<td>1.26</td>
<td>3.31 \times 2r_s</td>
</tr>
<tr>
<td>Intercrystalline failure (b)</td>
<td>1.26</td>
<td>3.31 \times 2rs</td>
</tr>
<tr>
<td>Intracrystalline failure</td>
<td>1.365</td>
<td>5.37 \times 2rs</td>
</tr>
<tr>
<td>Coupled failure</td>
<td>1.255</td>
<td>3.24 \times 2rs</td>
</tr>
</tbody>
</table>

A homebrew program, which is based on the flow chart in Figure 1, is used to calculate the fractal dimension values of the rock fractures shown in Figure 2 to 5(a). The extracted edge characteristics from those figures are shown in Figure 2 to 5(b). Then the following \( (\log_2 I, \log_2 N(F,I)) \) point sets and the corresponding fitting lines are evaluated and listed below:

Figure 9 Data set and fitting curve of granite tensile fracture

Figure 10 Data set and fitting curve of diorite tensile fracture
The actual calculated box dimension values of the four fractures are listed in Table 2.

**Table 2 Fractal dimension values of 4 rock fractures**

<table>
<thead>
<tr>
<th>Fracture Type</th>
<th>Fractal dimension by model analysis</th>
<th>Results from digital image processing and box dimension calculation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Granite, coupled failure</td>
<td>1.255</td>
<td>1.4731</td>
</tr>
<tr>
<td>Diorite, coupled failure</td>
<td>1.255</td>
<td>1.4044</td>
</tr>
<tr>
<td>White marble, intercrystalline</td>
<td>1.26</td>
<td>1.3762</td>
</tr>
<tr>
<td>fracture</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Diorite, intracrystalline fracture</td>
<td>1.365</td>
<td>1.5082</td>
</tr>
</tbody>
</table>

The calculation results and the values from theoretical analysis have a good agreement. Since the real rock fractures are almost never completely of a certain mode (intercrystalline, intracrystalline or coupled), plus the reason of the image resolution, the box dimensions and the results from theoretical analysis can't perfectly agree.

It can be said that the higher the box dimension a fracture has, the less likely the corresponding fracture mode will occur. From comparing Table 1 with Table 2, it can be found that the higher the box dimension a fracture has, the higher the $G_{c_{in}}$ value it'll have, and therefore the less likely the corresponding fracture mode will occur. Quantitative interpretation of it is given in this article using the concept of fractal.

**5. Conclusion**

The main procedures of digital image processing and fractal dimension calculation of rock fracture SEM photos, including gray-scale processing, low-pass filtering, Otsu's method and image binarization, edge detection and box dimension method, are given in this article. Using some real rock fracture SEM photos, the fractal dimension calculation results are agree with the theoretical analysis results, hence the algorithm and the program are considered correct.

The relationship between rock fracture fractal dimension and the fracture mode is also discussed in this article. The following conclusion is derived and confirmed with physical models: in the same grain size, the intercrystalline failure and the intercrystalline coupled with intracrystalline failure are more likely to happen, while the intracrystalline failure is less.
References


GROUT APPLICATION STATUS AND PROSPECT IN CHINESE SUBWAY CONSTRUCTION BY SHIELD METHOD

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In recent years, with the rapid development of China's economy, urban infrastructure has been developed vastly in China, especially the subway construction has bring great effect for Chinese travel. Due to the many advantages of shield construction method, China subway builders have chosen it as one of the main methods in the subway construction. As an important tache in subway construction in, the grouting is of great significance for subway construction. Especially, subway is mostly constructed in the ground of densely populated cities where settlement may lead to serious consequences, grouting for settlement control plays a crucial role. Therefore, China's subway builders have done a lot of research and practice about grouting. This paper gives a brief introduction to the subway shield method grouting material concept, function and classification, a total of 14 shield method subway construction in seven Chinese city have been analysed about grouting material, the application of grout of shield in China has been summarized, and the existing problems and its prospect has been put forward.

1. Introduction

In the 21st century, China began to accelerate the construction of the subway, China has 33 cities have been planning orbit transportation construction, including 28 cities already approved. It is estimated that by 2015, the whole of China metro operation and total mileage will amount to 3 000 km, and by 2020, there will be 40 urban construction the subway, the total planning mileage of 7000 km, is now 4.3 times in the total mileage[1]. With the domestic subway construction scale and area increase, because of its advanced technology, high mechanization degree and good control of settlement, shield method subway construction in China has become the mainstream of the subway construction methods. Grouting is a indispensability part of shield method in subway construction, grouting success is the key which can dicide the ground settlement control. Grouting material , the ratio of grouting material and the performance of the grouting stone body, are basic elements to success for grouting. This paper analyses recent shield method subway construction in China, summarizes the current application status of shield grouting, and the existing problems and prospect are discussed.

2. Shield grouting introduction

Grouting is an engineering method which need a complete set of mechanical equipment, adopt reasonable grouting technology, grouting material injection project object, in order to achieve filling, reinforcement, water plugging, lifting and rectification for engineering rock’ s and soil’ s
body. For shield method in subway construction, due to shield body shell and overcutting, a gap, exists between the shield skin and the surrounding ground. If the gap is not promptly filled, it will lead to the occurrence of ground settlement. The primary role of grouting is to fill this gap timely so as to control the ground settlement. In addition, there are the other two aspects for shield tunnel grouting: (1) grout consolidation forms a waterproof barrier around shield segment, as can enhance tunnel waterproof ability; (2) grouting can provide the stability of the tunnel early, make the integration of segment and surrounding soil, is helpful for shield driving direction control, and can improve the ultimate stability of the shield tunnel.

3. Shield grout classification

For the composition of the grout of shield, grout research can be divided into two branches: first, in Japan, the United States and other countries as a representative, shield tail grouting mainly adopts double-fluid grouting; the other one is single-fluid grouting in France, Germany and other European countries as a representative. Two kinds of grout have each advantage and disadvantage, and are used meanwhile at present in China.

(1) Single-fluid grouting, is generally made of fly ash, sand, cement, bentonite, water and admixture. The single-fluid grouting can be divided into inert grout and rigid grout. Inert serous is namely grout without cement material, the early strength and late strength of which are very low. Rigid grout is in the grout mixed with cement material, which has certain strength early and later. Condensation time of single-fluid grouting changes from a few hours to more than ten hours. Inert grout strength, initial set time and flow property are closely related to the water content. While water content is high, the strength is low and the pumping performance is well. Rigid grout strength, initial set time and pumping performance are closely related to water cement ratio. While water cement ratio is high, the strength is low and the pumping performance is well [2].

(2) Double-fluid grouting, is made of A liquid mixed by the cement mortar and B liquid generally being sodium silicate. According to initial setting time, double-fluid grouting is divided into slow junction of which initial setting time is 30 to 60 s, and transient setting type of which initial setting time is less than 20 s. Because coagulation time of double-fluid grouting is more shorter than single-fluid grouting, the grout can form effective filling around shield construction in short time, as is suitable for the bad geological condition. On the other hand, the short gelling time implies poor liquidity, which could cause plug grout pipe.

4. Statistics and analysis of Shield grout in China

During Shield methods being used and spreading in China, grouting materials, grouting method, grouting theory and so on have been explored more suited to the grouting process by combining foreign experience continuously. Rapid progress has been made, a lot of basic information and experience for future design, and construction has provided precious experience. According to the literature collected, shield grouting in the typical regions of China is analysed [3-7].
Table 1

<table>
<thead>
<tr>
<th>Project name</th>
<th>Geological conditions</th>
<th>Grout type</th>
<th>Grout material ratio</th>
<th>Initial setting time</th>
<th>Grout effect</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Test section engineering of Beijing metro line 5</td>
<td>Clay and sand at arch crown; sand pebble, and gravel at lower part</td>
<td>S</td>
<td>Host crystal is quicklime and flyash, auxiliary agent is ine sand, sodium base bentonite and water</td>
<td>10-12h</td>
</tr>
<tr>
<td>2</td>
<td>The zoo station - ShuangYuShu station of Beijing metro line 14</td>
<td>Silt at upper half of shield driving section; sand and pebble at lower half.</td>
<td>D</td>
<td>A liquid, water: cement: retarder: bentonite =400: 500: 5: 120; B liquid, 30 baume degree sodium silicate</td>
<td>In 20s</td>
</tr>
<tr>
<td>3</td>
<td>Section 9 of Beijing metro line 10</td>
<td>Pebble, round gravel, fine sand</td>
<td>D</td>
<td>A liquid, cement, water, flyash, bentonite; B liquid, 27-30 baume degree sodium silicate</td>
<td>30s</td>
</tr>
<tr>
<td>4</td>
<td>Some section of Shanghai metro line 1</td>
<td>Mollic epipedon</td>
<td>S</td>
<td>Bentonite: water: levigate flyash: black sand=1: 5: 3.4: 7</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>Section 10 of Shanghai rail transit line 6</td>
<td>Argillaceous sandstone with well stability. Ground water is general little except part</td>
<td>D</td>
<td>A liquid, water: cement: retarder: bentonite =800: 260: 0.9: 60; B liquid, sodium silicate</td>
<td>20-30s</td>
</tr>
<tr>
<td>7</td>
<td>The second grouting of some water-rich section in Guangzhou metro</td>
<td>Bedrock under river</td>
<td>D</td>
<td>Cement paste + sodium silicate</td>
<td>In 10s</td>
</tr>
<tr>
<td>8</td>
<td>Liyao station-Dashi station of Guangzhou metro line 3</td>
<td>Clay, silt, sand</td>
<td>D</td>
<td>A liquid, water, cement, retarder, bentonite, mud; B liquid, sodium silicate</td>
<td>12.5-13.5s</td>
</tr>
<tr>
<td>Project name</td>
<td>Geological conditions</td>
<td>Grout type</td>
<td>Grout material ratio</td>
<td>Initial setting time</td>
<td>Grout effect</td>
</tr>
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<td>----------------------------------</td>
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<td>---------------------</td>
<td>-----------------------------------------------------------------------------</td>
</tr>
<tr>
<td>9 Yue station-San station of Guangzhou metro line 2</td>
<td>Main hard rock, next quaternary soft rock with water, interbedded hard and soft rock</td>
<td>S</td>
<td>Sand, 50%~60%; water, 20%~25%; cement, 20%<del>25%; flyash, 7%<del>25%; bentonite, 0</del>2.5%; unit cement consumption, 100</del>200kg/m³</td>
<td>1.5-3h</td>
<td>Grout Can meet the actual construction need.</td>
</tr>
<tr>
<td>10 Hua station-Gang station of Shenzhen metro</td>
<td>Main hard rock, next quaternary soft rock with water, interbedded hard and soft rock</td>
<td>S</td>
<td>Bentonite, flyash, sand and water</td>
<td></td>
<td>The surface settlement of Shennan road was maximum 8 mm, the other data of detection conformed to the construction requirements</td>
</tr>
<tr>
<td>11 Huizhan center station of Shijifeng station of Shenyang metro line 2</td>
<td>Silty clay, medium and coarse sand, abundant water</td>
<td>S+D</td>
<td>S, water: cement: sand: flyash: Bentonite: swelling agent = 436: 130: 800: 500: 60: 54; D, Cement paste + sodium silicate</td>
<td>5-6h</td>
<td>The cumulative settlement of Su-Fu railway was 4.13 mm maximum.</td>
</tr>
<tr>
<td>13 Bingjiang station of Hangzhou metro</td>
<td>Muddy clay, high penetration silty fine sand, gravel</td>
<td>S</td>
<td>Water, lime, yellow sand, bentonite, flyash</td>
<td></td>
<td></td>
</tr>
<tr>
<td>14 Nanjing Yangtze river tunnel</td>
<td>Muddy clay, high penetration silty fine sand, gravel</td>
<td>S</td>
<td>cement: flyash: silty fine sand: Bentonite: water: water reducer = 60: 300: 1360: 80: 3: 350</td>
<td>6-12h</td>
<td>The abandon sand of Shield construction has been used in grouting.</td>
</tr>
</tbody>
</table>

For Grout type, S: single-fluid grouting; D: double-fluid grouting.
Statistical project includes 14 subway shield grouting engineering in seven cities. There are eight engineering in which single-fluid grouting has been used, grout material including cement, fly ash, bentonite, sand, water reducing agent, setting time is generally in 1.5-12 h that could be adjusted by different material ratio. Double-fluid grouting has been used in six other engineering, among which, single-fluid grouting was adopted in first grouting, and double-fluid grouting was used in secondary grouting at Huizhanzhongxin station - Shijiguangchang station of Shenyang metro line 2. The double-fluid grouting has been accelerated initial setting time by adding sodium silicate of certain baume degree mainly, B liquid. A liquid was also made of cement, fly ash, sand, bentonite, retarder and admixture generally. The setting time of the double-fluid grouting was few seconds to tens of seconds.

The single-fluid grouting was more used than double-fluid grouting slightly. For low construction cost, long setting time, low later strength, low later stiffness, grout easy running off, shield tail seal being required high, and grout pipe clear more easily, single-fluid grouting is more suitable for good geological conditions or undemanding ground settlement location. The single-fluid grouting fit most of needs of the project, such as Beijing line 5, Shanghai line 1, Guangzhou line 2, line 4, grouting effect is good.

The double-fluid grouting used in choice engineering was relative less. The equipment of double-fluid grouting is complicated, the cost is high, the later strength degree is high, and the grout is not easy to run off. On the other hand, grout pipe is easy to be blocked, sodium silicate could be harmful to person or pollute environment, and the demand to construction management is high. Double-fluid grouting is obviously advantageous for bad geological condition, large diameter tunnel and demanding ground settlement. Such as, for 10 section of double circular shield tunnel in Shanghai line 6, of which structure is complex and the diameter is large, immediate settlement can be controlled in 1 ~ 2 mm, and late settlement can be controlled within 0.5 mm/d. In section 9 of Beijing subway line 10, ground uplift and settlement are all less than 1 mm.

5. Discussion and prospect

During years of theoretical research and engineering practice, there have gotten considerable experience in shield grouting material selection and mixture ratio in China. They can provide feasible grouting design for most of complicated conditions, but for engineering science and technology innovation, there are still quite a few problems worthy to be discussed.

(1) Comparing with double-fluid grouting, most shortage of single-fluid grouting is setting time is long, as is not suit to be applied to bad geological condition and engineering complex situation. But its advantage is simple construction and low cost. If some additives can be found so as to make initial setting time shorten to dozens of seconds or even a few seconds, single-fluid grouting could replace double-fluid grouting in most cases.

(2) Sand is frequently chosen in most of single-fluid grouting, which generally need be accessed specially. If the sand dug out from shield could be used, it can reduce exhaust soil quantity and save the cost of grouting sand. Some scholars have do similar attempt [3].

(3) Grout condensation causes volume shrinkage, as is one reason of late ground settlement, so secondary grouting is often done. If volume shrinkage can be controlled, or it makes volume
expanding slightly, ground settlement will be control more well, and it can also reduce secondary grouting frequency. This problem can be solved by studying grout material ratio and admixture.

(4) Grout pipeline must be washed whenever grout is over for avoiding blocking pipe, as causes to waste grout material and flushing water, and pollutes the environment. If reasonable methods could be found to recycle waste, it will bring good effect in economy and environmental protection.

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THEORETICAL ANALYSIS AND ENGINEERING EXAMPLES OF GROUTING TECHNOLOGY IN GOVERNING TUNNEL SURROUNDING ROCKS

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Abstract: it is common to build tunnels and inclined shaft etc in railway engineering, hydraulic engineering and coal mine construction under circumstances of complex geological conditions in landslide areas and soft soil areas. These projects have characteristics of long tunnelling distance, shallow depth and high environmental protection requirements etc. In this paper, from the perspectives of high-pressure jet-grouting in governing landsides, reinforcement of mine inclined shaft surrounding soil and water culvert protection in soft soil layer, the mechanism of high-pressure jet-grouting is analysed, high-pressure jet-grouting mechanical model is established, and engineering examples are given at the same time.

1 Introduction.

For the high pressure jet grouting, high-pressure spray emitted from nozzle in drillings is used to impact and destroy the soil mass and at the same time solidified liquid with the spray is fully mixed with the destroyed soil to form concreted solid which plays a role of soil reinforcement and seepage proofing [1, 2, 8].

High pressure jet grouting can be applied in landslide treatment in railway project, reinforcement of tunnel surrounding rocks, reinforcement and seepage-proofing of surrounding soils of water tunnels in hydraulic engineering, and reinforcement and seepage-proofing of surrounding soils of mine inclined shaft, etc.

No matter in railway project, hydraulic engineering, or mine engineering, the purpose of grouting is to form arch rings with a certain strength by grouting inside the surrounding rocks of tunnels to bear the ambient pressure and reduce the pressure on tunnel linings. The concreted solid due to the grouting inside possible sliding soil mass might be able to improve the shearing strength of the weak surface and increase the stability of soil mass. Concreted solid by grouting is able to reduce the penetrability of soil mass and act a role as seepage-proofing and leaking stoppage [3, 6, 7].
2 Mechanical model of grouting reinforcement

Tunnel reinforcement section is as shown in figure 1, and its calculation diagram is as shown in figure 2.

![Diagram of grouting reinforcement](image1)

**Figure 1** sectional drawing of grouting reinforcement of tunnel

![Diagram of calculating thickness](image2)

**Figure 2** calculating diagram of the thickness of grouting reinforced zone of rock and soil layers

\[ b_k = a \left( \frac{1.55 \gamma H}{R_k} - 0.06 \right) \]

is the thickness of grouting reinforced zone on the top of the tunnel;

\[ b_B = a \left( \frac{1.27 \gamma H}{R_B} - 0.13 \right) \]

is the thickness of grouting reinforced zone on the both sides of the tunnel; \( R_k \) is the compressive strength of rock and soil on the top of the tunnel; \( R_B \) is the compressive strength of rock and soil at the two sides of the tunnel; \( b_\omega \) is the width of grouting reinforcement area, \( b_\omega = b_k - (b_k - b_B) \sin \omega \) \( \omega \) is the included angle between the joint plane and radius of the peripheral point of rock and soil.
The pressure of grouting reinforcement is:

\[ P = \frac{\left( z - \frac{2U_0^*}{3} \right)H}{z + 2Ga\left(B_y^* - E_y^{-1/3}\right)\sqrt{l-a}} \]

\( E_y \) is the elastic modulus of grouting reinforcement; \( B_y \) is the creep modulus of grouting reinforcement; \( G \) is the shear modulus of rock and soil; \( a \) is the calculating radius of tunnel; \( l \) is the radius of grouting reinforced zone; \( U_0^* \) is the displacement value;

The carrying capacity of grouting reinforcement is:

\[ q_y = R_y \left( \frac{l - \frac{a^2}{l^2}}{2} \right) \]

\( R_y \) is the Compressive strength of grouting reinforcement.

Therefore, grouting parameters could be determined according to the required intensity of grouting reinforcement [4, 5, 9].

3 Engineering Practice

3.1 Reinforcement of Sliding Mass of Xiangyang-Chongqing Railway Tunnels by Grouting

Chaijiapo Tunnel of Xiangyang-Chongqing Railway is in length of 230m, running across new sliding mass formed by the aggregate of the old sliding mass. The top-down stratum is respectively recent slop wash, old landslide aggregate, and earlier slop wash. The recent slop wash is with bronzing and isabelline clay in the upper part and off-white clay in the lower part, including isolated diorite; old landslide aggregate is composed of mica quartz schist, chlorite schist and deteriorated diorite among which the chlorite schist would disintegrate into green earth in the presence of water and thus roof falling and collapsing would come into being without immediate support; the earlier slop wash is with sepia and luidity clay in the upper and chlorite schist in the lower. The tunnel under operation and the one under construction are respectively cut by two sliding surfaces which are clay sliding zones in thickness of 1m according to the exploration. The tunnel under operation was completed in 1970s and upon the completion the deformation had been in ceaseless increasing. The stability of the tunnel is ensured after numerous reinforcement treatments. The center distance between the newly built tunnel and the one under operation is 26-28m. The construction commenced in March, 1993. During the construction, there had happened 3 times of collapsing which gave rise to the severe deformation of the one under operation. The maximum width of vault crack was up to 80mm, and the lining was cracking and chipping, and thus the driving safety in the tunnel was under great threat. As a result, it was forced to shut down in May, 1945. Later monitoring was conducted for the tunnel under operation and slip mass.

It is decided to take grouting for concreting by the analysis on mechanical models of the new tunnel and the one under operation. In order to ensure the normal construction of the new tunnel, concrete solid with certain strength comes into being between the new and old tunnel by grouting.
in sliding soil mass to reduce the pressure on lining. The construction sequence is as follows: first, to reinforce the new tunnel and the surrounding soil mass (first phase); second, to reinforce the soil mass between the two tunnels (second phase), and third, to reinforce the tunnel under operation (third phase). And the grouting parameters are as follows: reinforcing ring by grouting concreting around the old and new tunnels is 4m in thickness; reinforcement depth of soil mass between the old and new tunnels is 10m. Grouting pressure in the first phase is 2.0-3.5MPa and that in the second phase is 0.2-1.5MPa. For the grouting in the third phase, when the final pressure of grouting is up to 2.0MPa, as compression deformation of soil mass is generated under grouting pressure, the surrounding deformation of tunnels due to grouting in soil mass is obviously inferior to that due to the grouting in rock mass. When the final grouting pressure on the section with support by steel arch centering is up to 2.5MPa, the maximum grouting pressure would be up to 2.0-2.5MPa when the complete lining is without support, and there is no deformation to the new tunnel which indicates that the relationship between the maximum grouting pressure and deformation has much to do with the characteristics of the injected medium.

After grouting and sampling by drilling, the drill core is with ultimate compressive strength of 8.5MPa. Upon the commencement of tunnel excavation, it is noted by inspection that the joint plane and fissure plane within the stratum are with grouting concreted solid in general thickness of 1-3mm. In the bottom half section, upon commencement of excavation, it is observed that there is grouting concreted solid in different thickness (the maximum thickness is about 400-500mm) in the sliding surface. The strength of soil mass in sliding surface is improved. Indoor soil tests are conducted by sampling after the tunnel excavation and it is measured that: water content is w=7%-9%; cohesion is c=23kPa; angle of internal friction is $\phi=20.8^\circ$-$38^\circ$. By taking samples from grouting concreted solid within the sliding zone to do direct shear test, it is measured that: (1) soil with fine grouting, $c=106kPa$, $\phi=22.8^\circ$; and (2) soil with thick grouting, $c=156kPa$, $\phi=22.8^\circ$. It follows that the cohesion of sliding zone increases 6.5 times, the cement grout of collapsing section has made the collapsed soils and rocks and so on in cementation.

3.2 Grouting Reinforcement and Sand-drift Layer on Main Inclined Shaft of Tiebei Mine

Tiebei Mine is located in the Northwest of Inner Mongolia Zhajiruoer coal field, and the coal-bearing formation consists of sandstone, silt rock, argillaceous sandstone, mudstone and coal seam, and the coal seam fracture growth is good aquifer and permeable layer. To make the inclined shaft pass through the sand layer safely, through comparison between technical and economic analysis and several treatment methods, high-pressure jet-grouting method is used for processing.

See figure 1 for the cross-section of inclined shaft (horizontal dip of 15), and vertical section of the inclined section is as shown in figure 3. The purpose of high-pressure jet-grouting method is to form a closed jet-grouted solid around the inclined shaft, and the key of sand-drift layer is that whether pile group is able to form a closed entirety in sand-drift layer, with requirement of being seamless among piles and higher requirements on jet grouting.
Considering the influence of groundwater on the pile diameter and the decline possibly produced by the drill hole, pile groups are combined according to plum-shaped hole arrangement. According to results of site injection test, the pile diameter of the design is determined as 0.96m, center distance of the pile is 0.8m, and the line spacing is 0.72m. The plum-shaped center pile is injected in the last, stop lifting every 0.3m lifting and make directional jet grouting in six directions for one minute and rotated injection in situ for one minute to strengthen the connection between piles. After treatment, very good plasma soil jet-grouted solid is formed around the shaft. The quick sand in the shaft could be dug up completely, and the rotating body inside the shaft could be completely exposed. When the shaft is under condition of underground water depth of 0.5~0.9m, there is no quick sand and water leakage, reaching the purpose of Inclined Shaft Sand-drift Layer.

3.3 Horizontal Grouting for Xinle Culture Station of Shenyang Subway Line No. 2

Xinle Culture Station is located in north to the T-junction of Huanghe North Street and Longshan Road, in north-south arrangement along Huanghe North Street. It is island-type station with two underground floors, of which the main structure is reinforced concrete structure of single arch. It pioneers the application of new pipe roofing method-undermining method for construction. The station is with total length of 179.8m, standard section's width of 26.2m, height of 18.9m, earth covering in the top of standard section structure of 7.6m-11.2m, and baseboard burial depth of 26.5m-30.1m.

The terrain of the site is even with ground elevation of 47.50-51.90m. According to the geologic report, the vault of the station is basically composed of miscellaneous fill and silty clay with partial medium-coarse sand and with cavities in individual lots. Thus it is with poor geological conditions and tends to have collapsing.

In order to ensure the soil stability during the pipe jacking, to avoid water losses and soil erosion, and to decrease the harmful effect on pipelines by sedimentation, advance pre-grouting will be conducted on the working face in the front of pipe-jacking with grouting depth of 1.5m and jacking length of 1m.

(1) Grouting Scope and Grouting Holes Arrangement

For pipe-jacking and grouting cycle operation, the concreting zone for each cycle of water sealing by injection is with length of 1.5m and jacking length of 1m. The grouting scope is as long as 3m in longitudinal direction, the first 1.5m is water sealing and reinforcement zone, and the forward 1.5m is designed as extensive reinforcement zone in order to ensure the soil stability of working
face. The grouting holes are in uniform arrangement along pipelines, of which the number is 8 and extrapolation angle is 5°-10°. See Fig. 4 for the arrangement of grouting holes.

Figure 4 drawing of combs of grouting

(2) Grouting Materials
Grouting material is prepared by cement, sodium silicate and bentonite. Grouting with bentonite shall be 24 hours in advance.

(3) Grouting Pressure
Cohesionless soil is of 0.3-0.5MPa and cohesive soil is of 0.6-1.2MPa, and the grouting amount will be determined by grouting tests.

A certain length of soil mass will be maintained in the pipeline before grouting for the generation of soil plugs. According to the inspection upon actual excavation, grouting effect is favorable, and the soil mass is stable without leakage of underground water.

4 Conclusions
1 With the landside soil creep, use the deformation amount to control grouting pressure and then to control grouting rate and grouting amount; the establishment of a series of strict measures such as monitoring system is essential to the success of grouting.

2 The comprehensive adoption of filling grouting, high pressure jet grouting, and advance pre-grouting and so on can ensure a favorable reinforcement effect.

3 Trial tunneling and running of equipment system are adopted for the fast tunneling achievement in a soon way. Advance geology report and the employment of advance reinforcement for surrounding rocks are typical for the fast and smooth crossing of TBM over sectors with defective geological environment.

4 Monitoring and inspection are important for the dynamic design and information-based construction of tunnels. And the monitoring system ensures the information platform for the construction of grouting form monitoring, management, and direction and so on.
References

NUMERICAL SIMULATION ON STABILITY OF REINFORCING BARS FOR SLICING DRIFT DURING UNDERHAND CUTTING AND FILLING OF PANEL

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Abstract: Aiming at the backfill quality improvement in a mine, the numerical model of the slicing drift and hexagon stope in the under cutting and filling is built by the FLAC 3D software. The artificial roof of the slicing drift with the ordinary backfill and reinforcement backfill is investigated. Then the slicing drift and hexagon stope stress field, the plastic field and displacement field is analyzed quantitatively. The mechanical parameters of the reinforcement backfill are obtained by the equivalent strength principle from the reinforced concrete design standard. The analytical results showed that, the artificial roof of slicing drift against the stope safety production with ordinary backfill. The caving and discrete block falls are prone to be happened in the path and slicing drift intersecting channels. Therefore, monitoring and the reinforcement density should be strengthened in the key areas. But the density of the reinforcement should be reduced in the surrounding rock where only the slicing drift is excavated.

The total stope district, the ore production and grade are all reduced as the increase of the mining depth. If the stope strength in unit area is unchanged, the ore output could not be improved in the limited operating area and the ore production may be reduced and hard to remain the same as before. In all, it is inevitable to expand the stope section and improve efficiency work by mechanization matching operation. Longshou mining district, as the earliest mining in Jinchuan group ltd., has always being paid attention to scientific mining and exploring actively by new mining technology and methods suitable for the distribution condition of metal mining. The slicing drift and hexagon stope method was introduced by Jinchuan group Ltd. based on many years’ test and practice. The method is a new style effective method which satisfies the need of safety and production. It changes the loading condition of the stope route and develops the stability of the stope.

1 The method of slicing drift and hexagon stope

Figure 1 presents the method of slicing drift and hexagon stope in the under cutting and filling. The stage and slicing is 60m and 15m in height. The size of the stope is 75m in length and it is the same as the orebody level thickness in width. The stope is top-down type in the stage. Several panel district are stoped at the same time and there is no pillar among them. The pulse internal and external combined mining quasi system is introduced in the method of slicing drift and hexagon stope. The slicing ramp is set in the surrounding rock of hanging wall, semicircle arch of straight wall and supporting of sparging and anchoring net. A slicing drift is set when the stope is dropped by 15m. The sublevel drift, connected with the ramp, is set at the pulse external of the hanging wall. The orebody can be entered by the slicing connection channel dipped in the sublevel drift. The sublevel drifts are channels for the equipment, power, material, people in and out, rubble and poor and rich orebody chosen and each one of them can
serve 5 layer. The slicing drift is set at the center of the panel district and the size of the section is 4.5m or 5.2m in width and 3m in height. A pulse out ore pass is set beside the slicing drift at the hanging wall. The air shaft, 1.2m in diameter, is reserved as the return air with the filling road in the slicing drift at the footwall. Two or three air shafts are chosen in the slicing drift according to the need of stope route length and the maximum filling. The waste well is set for the waste rock of three or four panel district beside the pulse external slicing drifts. The traverse pulse filling drifts are set at both sides of panel district and its section area is 2m plus 2m. The stope is dropped by 15m. The filling well is connected with end of the stope route. At the same time, the air shaft is set and the traverse pulse filling drifts are connected with the stage along pulse ones.

In this method, the slicing drift is the aiming drift. Because the backfill is the direct roof for the mining drift, it can be called as the artificial roof. Its stability is the key to the success of the under cutting and filling mining technic. As a result, the analysis of slicing drift stability is needed for the design and the production. In this paper, combined with the scene, the numerical model of the slicing drift and hexagon stope in the under cutting and filling is built by the FLAC 3D software and the stability of the reinforced filling body artificial roof is analyzed.

![Fig.1 The mechanized mining method with hexagonal filling route](image)

**2 Numerical modeling**

Modeling was carried out with FLAC3D which is based on the finite difference numerical method with the Langragian calculation method. Finite difference method, in which the continuum is represented by a series of discrete grid point, can be better applied to modeling of large deformation rock mechanic problems. The displacement field is computed by approximation the differential equations for the system as a set of different equations that are solved discretely at each grid point. Otter et al. (1966) described that the lumped mass at the grid point for each point in space at a new time that is given by
the old time plus some time increment, called as time step. The acceleration is integrated over the time step to give grid points velocity and displacement. The stress is transformed to the new grid point force. The process is repeated for all the grid points in the system until a new state of equilibrium is achieved (Singh et al., 2010).

2.1 Mesh and boundary conditions

Due to computer running time and capacity restrictions, the numerical area is taken as 5 times as stope. Therefore, the face length was taken as 720m on the +x coordinate axis in the model. The size of the width and height are taken as 144m and 140m on the +y and +z coordinate axis in the model. In order to obtain more precise stress distribution results, small mesh size was selected. The model was composed of 456808 elements and 480600 grid points. The load is applied from the depth of the model to the face. The side of the model is applied roller type restriction and the bottom of the model is confined at the three directions. The slicing drift and the hexagon stope are presented in Figure 2 and Figure 3.

2.2 The mechanics model and the ground stress

Orebodies of the Longshou mining district are in initial stress conditions. The buried depth of the stope is deep and the horizontal stress is big. Most of the joints and the fracture are distributed randomly in a closed state. In a whole, the rock, rich ore, poor ore and backfill can be considered as an isotropic, elastic and plastic continuum medium. In the current study, the constitutive model used for the backfill is an elastic and perfectly plastic model. For the ore, a strain softening constitutive model is adopted. The Mohr-Coulomb failure criterion is used.

According to the in situ data of ground stress in Jinchuan mining district, the principal stress is close to horizontal direction and the dip is less than 10 degree. The ground stress rule is given as follows:

The maximum principal stress regression equation is: $\sigma_1 = \sigma_{h_{\text{max}}} = 0.098 + 0.05068H$(MPa)

The minimum principal stress regression equation is: $\sigma_3 = \sigma_{h_{\text{min}}} = 0.015 + 0.0200H$(MPa)

The vertical principal stress regression equation is: $\sigma_v = 0.208 + 0.02542H$(MPa)

2.3 Selection of the material parameters

The material parameter is one of the key factors for the numerical simulation. The basic property tests were carried out using the in situ specimens. Table 1 presents the mechanical parameters of rock mass
in Longshou mining district (S. H. Wang et al., 2004)

The reinforced backfill is a composite material composed by steel mesh and backfill. It is a heterogeneity and nonlinear material. If the separation model considering different materials is taken to the element simulation, the calculation is too big to analyze the whole structure. Therefore, it is assumed that the bond between the steel mesh and backfill is well and the slip between them can be ignored. They are as a whole in the equivalent model. According to the strength equivalent principle, the tension strength of the concrete material which contains the reinforced backfill is defined in the following function,

\[ R_t = R_{t,\text{concr}} \left( 1 + \frac{R_{y,\text{steel}}}{R_{t,\text{concr}}} \frac{V_{\text{steel}}}{V_{\text{steel}+\text{concr}}} \right) \]

where \( R_{t,\text{concr}} \) is the dynamic tensile strength of the backfill; \( R_{y,\text{steel}} \) is the dynamic yield strength of the reinforcement; \( \frac{V_{\text{steel}}}{V_{\text{steel}+\text{concr}}} \) is the volume ratio of the reinforcement in the backfill structure.

The equivalent elastic modules is adopted by the elastic modules of the reinforced backfill,

\[ E_e = \frac{(A_t E_{\text{concr}} + A_{\text{steel}} E_{\text{steel}})}{A} \]

where \( E_{\text{concr}} \) is the elastic modules of the backfill; \( E_{\text{steel}} \) is the elastic modules of the reinforcement; \( A \) is the total area of the longitudinal profile of the reinforced backfill; \( A_t \) is the area of the backfill; \( A_{\text{steel}} \) is the area of the reinforcement.

<table>
<thead>
<tr>
<th>Name</th>
<th>Elastic modulus(Gpa)</th>
<th>Poisson's Ratio</th>
<th>Density (g/cm³)</th>
<th>Friction Angle (°)</th>
<th>Tension strength(MPa)</th>
<th>Cohesion (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Peridotite</td>
<td>15.25</td>
<td>0.18</td>
<td>2.930</td>
<td>37</td>
<td>0.7</td>
<td>2.875</td>
</tr>
<tr>
<td>Marble</td>
<td>16</td>
<td>0.16</td>
<td>2.86</td>
<td>39.62</td>
<td>1.1</td>
<td>1.848</td>
</tr>
<tr>
<td>Orebody</td>
<td>13</td>
<td>0.23</td>
<td>3.024</td>
<td>36.85</td>
<td>0.85</td>
<td>2.255</td>
</tr>
<tr>
<td>Filling</td>
<td>7.28</td>
<td>0.21</td>
<td>1.716</td>
<td>36.6</td>
<td>0.6</td>
<td>0.76</td>
</tr>
</tbody>
</table>

Fig.3 confirmation of the cohesion \( c \) and friction \( \phi \) of the concrete
In the current study, the reinforced backfill is equivalent to the high strength backfill or concrete with a certain strength. The cohesion and friction of the concrete is predicted by the tension and compressive design strength based on the Mohr-Coulomb criterion. Then the values are compared with that obtained from the formula which contains the elastic modules and the tension strength of the reinforced backfill. The confirmation of the cohesion and friction of the concrete is presented in Figure 3. Table 2 presents the value of the concrete cohesion and friction. The mechanical parameters of the reinforced backfill are presented in Table 3 (X. Li et al., 2004).

### Table 2 The cohesion and friction of the concrete

<table>
<thead>
<tr>
<th>Concrete grade</th>
<th>Cohesion (MPa)</th>
<th>Friction (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C15</td>
<td>1.279</td>
<td>50.85</td>
</tr>
<tr>
<td>C20</td>
<td>1.625</td>
<td>52.598</td>
</tr>
<tr>
<td>C25</td>
<td>1.944</td>
<td>53.817</td>
</tr>
<tr>
<td>C30</td>
<td>2.261</td>
<td>54.90</td>
</tr>
</tbody>
</table>

### Table 3 Mechanical parameters of the reinforced backfill

<table>
<thead>
<tr>
<th>Reinforced backfill</th>
<th>The value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastic modulus(Gpa)</td>
<td>7.624</td>
</tr>
<tr>
<td>Poisson’s Ratio</td>
<td>0.21</td>
</tr>
<tr>
<td>Tension strength(MPa)</td>
<td>0.89</td>
</tr>
<tr>
<td>Friction Angle (°)</td>
<td>34.732</td>
</tr>
<tr>
<td>Cohesion (MPa)</td>
<td>1.251</td>
</tr>
<tr>
<td>Volume modulus(Gpa)</td>
<td>4.3816</td>
</tr>
<tr>
<td>shear modulus(Gpa)</td>
<td>3.1504</td>
</tr>
</tbody>
</table>

### 3 Numerical modeling results

The deflection and dangerous points are mainly distributed in the geometric center based on the thin plate theory. The monitoring points are set at the geometric center of the slicing drift artificial roof to monitor the difference between the stress and displacement of the reinforced backfill and ordinary backfill. The excavation and filling of the slicing drift and the hexagon stope are simulated as one step.  

#### 3.1 Reinforcement influence for the stress field of the slicing drift artificial roof

The maximum principal stress and time step curve is presented. It is shown that whether there is reinforcement or not in the artificial roof, the maximum principal stress is in the range of -14MPa to -1MPa. It was dropped sharply to 2MPa at step 2. That is because release of the stress which is reduced by the backfill undertaken after the filling of the slicing drift and the stope. It can be observed that the stress of the reinforced backfill artificial roof is lower than the ordinary backfill. It is meant that the integrity of the backfill is kept well by the mechanism of the reinforcement.

The minimum principal stress and time step curve is presented. The tension stress appears in the excavation step 1 and it is reduced as the following filling and excavation. The compression stress appears in step 4 for the isolation effect of the backfill. The tension of artificial roof with the ordinary backfill is 0.62MPa while the permissive tension value is 0.6MPa. But the tension of roof with reinforced backfill is 0.69MPa while the permissive tension value is 0.89MPa. It is shown that the tensile failure or even caving and dropping blocks may be happened in the artificial roof if the exposed slicing drift area is too big.

The vertical principal stress and time step curve is presented. Just as the same as in Figure 5, the tension stress of the artificial roof with the ordinary backfill exceeds the permissive tension value. Therefore, the monitor and support of these area should be paid more attention. Although there is tension in the reinforced backfill, the stress value and the tension area is reduced for the effect of reinforcement. As it is presented in the three figures, the three principal stresses show the similar trend.
rule.

3.2 Reinforcement influence for the plastic field of the slicing drift artificial roof

The strength and capacity of rock mass is reduced in the plastic state. The plastic deformation and the distribution state of the backfill artificial roof is one of the important factors in the rock mass stability analysis. Figure 4 and Figure 5 present the plastic area of the slicing drift and the hexagon stope.

When the slicing drift is excavated, the tension area of the reinforced backfill artificial roof is smaller than that of the ordinary backfill one. As the formation of the excavated hexagon stope, both the tension areas are increased. A breakthrough plastic area is appeared in the ordinary backfill artificial roof and a continuum tension failure happened in the artificial roof direction. When the big section intersecting tunnels is formed, the caving happened in the artificial roof for the influence of the tension and shear failure. It is bad for the mining production. When it is reinforced, the tension value and area is reduced and the artificial roof stability is improved. Although there is big plastic area in the top of the artificial roof, the artificial roof is protected by the stability structure which is formed by the reinforcement and the backfill. As the following filling and excavation on next floor, the plastic deformation area is reduced. Therefore, the reinforcement should be subtracted in the slicing drift of the surrounding rock and the stability area.

3.3 Reinforcement influence for the displacement field of the slicing drift artificial roof

After the excavation of the slicing drift and the hexagon stope, the displacement deformation happened in the surrounding rock and a deformation loose circle is formed. The bigger the goaf, the bigger the loose circle. It is necessary to monitor the displacement in the geometric center of the artificial roof and
the surrounding rock slicing drift. The trend of the displacement subsidence is similar in the two line graph. After the excavation of the slicing drift and the hexagon stope, the maximum displacement subsidence is 14.5mm for the ordinary backfill while it is 7.5mm for the reinforced artificial roof. The subsidence is smaller in the surrounding rock because the slicing drift is the only excavation drift. After the filling of the slicing drift and the hexagon stope, the subsidence is reduced. The displacement is declined fast in the excavation step 3 and it is nearly stable in the excavation step 4. The reason of it is that the displacement and the deformation is limited by the compacted and consolidated medium and the reduced deformation loose circle. In order to keep away the caving and discrete block falls in the intersecting tunnels, the steel mesh should be added at the big goaf. But it can be subtracted in the slicing drift of the surrounding rock.

4 Conclusions

In this study, the numerical code FLAC has been applied to simulate the stability of slicing drift of panel underhand cutting and filling. Based on the results from the simulations, the following conclusions are drawn:

(1) For the reinforced or ordinary backfill in the artificial roof of the slicing drift, based on the analysis of stress field, plastic field and displacement field, the ordinary backfill is not good for the mining safety production. The density of the reinforcement should be increased in case of caving and discrete block falls happening in the path and slicing drift intersecting channels. However, in the surrounding rock where only the slicing drift is excavated, the density of the reinforcement could be reduced case by case.

(2) For the exposed rectangular plate formed by excavated in the layered tract, the tensile failure is started from the center which is the position of the maximum deflection. Therefore, it is necessary to strengthen the artificial roof supporting and monitor the middle position.

(3) For reducing the exposed length of the slicing drift route, the strengthened point pillar artificial roof should be set in every certain distance. Then the tension stress will be reduced and the slicing drift can be in a stability state.

(4) For backfill, the quality is important to the mining safety production. It should be paid more attention to ensure the strength index.

Acknowledgement

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References

STUDY ON SOFT SOIL FOUNDATION TREATMENT OF PIPELINE PROJECT

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China Construction Fifth Engineering Division Corp. LTD

When the pipeline drills through long distance under mucky silt clay conditions, the low bearing capacity of foundation will lead to construction difficulties, even other catastrophic issues, such as ground surface collapse. Therefore, the rational construction scheme has profound significance for the pipeline project resting on soft soil conditions. With reference to second phase water supply project in Changxing Island Port Industrial Zone, the laboratory experiments were carried out to obtain the underlying soil’s property parameters. Through statistical analyses of more than 80 physical property experiments of mucky sily clay, the empirical formulations of natural water content and void ratio, of unit weight and void ratio, and of liquid limit and plastic index have been defined, which can be utilized to provide theoretical basis for foundation treatment scheme. Through scheme comparison and selection, light well point dewatering system, dumping rockfill and extruding silt methodology were suggested. The closed-form solutions to calculate the height of the rockfill, the deposition inside the mud and the height above mud surface have been developed. The software package FLAC3D was used to perform numerical analysis of the foundation bearing capacity and corresponding settlement for five rockfill height conditions (1.00, 1.25, 1.50, 1.75 and 2.00 m). The final decision of rockfill height was determined based on the numerical simulations. Finally, 12 points of heavy dynamic sounding test were conducted to collect the foundation bearing capacity data after the implementation of rehabilitation proposals to verify the effectiveness of the developed formulations.

1. INTRODUCTION

The infrastructure of sewer, gas, water and energy pipelines stretches out under almost every part of urban China. More than 93,000 km long gas pipelines are in service currently. The total length of water and sewage pipelines is not readily available. Among these pipelines, at least 1/10 were installed more than 40 years ago. They have already reached the end of their useful life spans and are in urgent need of replacement. The Chinese government estimates that over the next 10 years, over 50 trillion CNY must be spent to repair and doubled pipelines are going to be installed to solve the resources imbalance issue, such as water resources.

Following water transfer projects have been built to cope with water imbalance challenges: Mopanshan Long-distance Water Transfer Project in Harbin (Zhao, 2008), Dahuofang Reservoir Long-distance Water Transfer Project in Shenyang (Dong et al., 2004), South-to-North Water Diversion Project (Wei, 2005), Luanhe River Diversion Project in Tianjin and Tangshan, Water Diversion Project from Yellow River to Qingdao City, Water Diversion Project from Yellow River to Shanxi Province, Water-transferring Project from the Datong River to the Qinwangchuan Irrigated Area, Water Diversion Project from Dong River to Shenzhen City, and Longmentan Water Transfer Project in Quanzhou City, etc. The modern development of water division project leads its objective to industrial and living utility instead of agricultural and flood control requirements. The project scale becomes larger with the increased construction difficulties. In this paper, the water source of second phase water supply project in Changxing Island Port Industrial Zone is imported from Dahuofang Reservoir. 165 km long water pipelines, connected from Anshan pump station, will lead the water to Biliu River Reservoir. Finally,
the water will be distributed to Jinzhou and Wanfangdian area of Dalian City.

The constructed pipelines drill through long distance under complex soil conditions. In order to transport large quantity of water, large diameter pipeline was selected, which has the potential threaten induced by the lack of bearing capacity of foundation resting on the soft soil (Shin et al., 2008). The mucky silty clay condition results in low bearing capacity of foundation and corresponding construction difficulties. For specific locations, even other catastrophic issues such as ground surface collapse may happen. There are a certain bonding effects existing between mucky silty clay particles. Before excavation, mucky silty clay is in the soft plastic state possessing bearing capacity. However, its structural formation will be destructed once the excavation implemented, such that it will be in flow state and lose bearing capacity (Rowe & Skinner, 2001; Bergado & Teerawattanasuk, 2008). The ground surface collapse may happen accompanying with threaten to human lives and direct economic losses. This paper addresses the investigation of widely distributed soft soil condition in Wanfangdian area by laboratory tests. The settlement pattern of foundation resting on mucky silty clay has been studied and the theoretical framework of proper rehabilitation strategies is proposed for the pipeline construction project. Numerical simulations are performed to verify the close-form solutions and parametric analysis has been done to optimize the foundation strengthening schemes.

2. PROJECT BACKGROUND
The second phase water supply project in Changxing Island Port Industrial Zone is located in Wanfangdian area of Dalian City. The head of the pipeline construction is starting from Dongfeng Reservoir connected by DN1800 steel pipe. The toe is connected to the water treatment plant. The waste streams are divided in Santai Town to serve Changxing Island Port Industrial Zone and Jiaoliu Island Industrial Zone respectively. The water source is coming from second phase of Dahuofang Reservoir project after pumping in Anshan pump station. The total length of the pipeline network is 165 km, where the discharge of 0.288 billion m3/year is designed.

The total length of pipeline construction site is 74.32 km composed of 49.86 km in Changxing Island and 24.46 km in Jiaoliu Island respectively. For the Changxing Island site, the connection pipeline network is 35.73 km long using 2×DN1400 steel pipes, 6.40 km long 2×DN1200 steel pipe is built outside the island, and 7.73 km long 2×DN1200 steel pipe is built inside the island. Jiaoliu Island project adopts 2×DN900 ductile iron pipes, 2×DN900 steel pipes and 2×DN1200 steel-plastic composite pipes with the length of 23.31 km, 623 m and 520 m respectively.

In this paper, the stake number between 33+406~39+350 with the length of 5.944 km is investigated in terms of laboratory experiments. The studied part is located in Qingshan Village and Xiangxi Village, Santai Town, Wanfangdian City. According to the exploration survey, the soil layers are determined from the ground surface as follows: 0.5-2.8 m silt is distributed widely; 0.5-3.8 m silty clay in plastic state is distributed between the stake number of 33+406~34+990; 2.9-9.0 m fine sand with 20% gravel is distributed between the stake number of 33+805~38+310 and 38+550~39+390, where the cumulated length is 4.30 km; 0.5-7.5 m mucky silty clay is distributed below above layers, where the bearing capacity is measured as 35 kPa; 0.5-6.5 m medium-fine sand with 20% gravel is distributed in the bottom.

The water level is about 0.00-2.50 m in depth, such that the pipes in this part are below the water level. According to the criterions in GB50487-2008 (2008) and GB50021-2001 (2009), the ground water is weak erosive for steel pipes. From the empirical estimation, the permeability coefficient of fine sand layer under the ground water level is between 0.008-0.01 cm/s, of fine sand layer in the middle depth is around 0.01-0.02 cm/s, and of the mucky silty clay layer in the bottom is within the range of
0.22×10^{-7}-4.31×10^{-7} \text{ cm/s.}

### 3. LABROTARY EXPERIMENTS

GB50021-2001 (2009) recommends stochastic modeling methodology (Vanmarcke, 1977) to statistically calculate the parameters of geomaterials. The anisotropy and heterogeneous of soil can be characterized well. The spatial distribution property of soil parameters can be obtained by integration:

$$X_h(z) = \frac{1}{h} \int_z^{z+h} X(z)dz$$  \hspace{1cm} (1)

where, \( h \) is the depth of soil layers, \( z \) is the variable along the depth of soil layers.

Following expressions can be retrieved from Equation (1):

$$\mu_v = E[X_h(z)] = \mu$$  \hspace{1cm} (2)

$$\sigma_v^2 = \text{Var}[X_h(z)] = \sigma^2 \Gamma^2(h)$$  \hspace{1cm} (3)

$$\delta_v = \frac{\sigma_v}{\mu_v} = \delta \Gamma(h)$$  \hspace{1cm} (4)

where, \( \mu \), \( \sigma^2 \) and \( \delta \) are the mean, standard deviation and coefficient of variation of soil parameters respectively; \( \mu_v \), \( \sigma_v^2 \) and \( \delta_v \) are the mean, standard deviation and coefficient of variation of soil spatial parameters respectively; \( \Gamma^2(h) \) is the reduction factor of standard deviation.

Regression method can be used to fit the field measurement data. The laboratory experimental results are summarized as scatters, and the regression equation is in the form of \( y = a + bx \). In this regard, least square method can be used to determine the coefficient of regression equation.

$$a = \frac{\sum_{i=1}^{n} y_i - b \sum_{i=1}^{n} X_i}{n}$$

$$b = \frac{\sum_{i=1}^{n} X_i y_i - \frac{1}{n} \sum_{i=1}^{n} X_i \sum_{i=1}^{n} y_i}{\sum_{i=1}^{n} X_i^2 - \frac{1}{n} (\sum_{i=1}^{n} X_i)^2}$$  \hspace{1cm} (5)

where, \( \bar{X} = \frac{1}{n} \sum_{i=1}^{n} X_i \) and \( \bar{Y} = \frac{1}{n} \sum_{i=1}^{n} y_i \).

The second phase water supply project in Changxing Island Port Industrial Zone drills through saturated mucky silty clay layers in several locations. The physical property parameters of soil from more than 80 physical property experiments are tabulated in Table 1.

Table 1. The physical and mechanical properties parameters of mucky silty clay

<table>
<thead>
<tr>
<th>Property</th>
<th>Min</th>
<th>Max</th>
<th>Mean</th>
<th>Std</th>
<th>Cov</th>
<th>Upper</th>
<th>Lower</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water content w %</td>
<td>70.14</td>
<td>41.52</td>
<td>52.85</td>
<td>4.98</td>
<td>0.094</td>
<td>53.52</td>
<td>52.18</td>
</tr>
<tr>
<td>Density γ kN/m3</td>
<td>18.9</td>
<td>17.1</td>
<td>17.59</td>
<td>0.528</td>
<td>0.03</td>
<td>17.57</td>
<td>17.52</td>
</tr>
<tr>
<td>Specific gravity Gs</td>
<td>2.741</td>
<td>2.719</td>
<td>2.739</td>
<td>0.014</td>
<td>0.005</td>
<td>2.741</td>
<td>2.736</td>
</tr>
<tr>
<td>void ratio e</td>
<td>1.427</td>
<td>1.033</td>
<td>1.257</td>
<td>0.136</td>
<td>0.108</td>
<td>1.277</td>
<td>1.237</td>
</tr>
<tr>
<td>Liquid limit wL %</td>
<td>48.9</td>
<td>31.2</td>
<td>38.92</td>
<td>4.09</td>
<td>0.126</td>
<td>39.57</td>
<td>38.27</td>
</tr>
</tbody>
</table>
The empirical formulations of natural water content $w$ and void ratio $e$, of unit weight $\gamma$ and void ratio $e$, and of liquid limit $w_L$ and plastic index $IP$ have been defined, which are presented in Fig.1, 2, along with the scattered experimental data.

**Figure 1.** The relationship between water content $w$ and void ratio $e$

**Figure 2.** The relationship between unit weight $\gamma$ and void ratio $e$

### 4. FOUNDATION TREATMENT

Pipeline project is similar as other infrastructure project starting from trench excavation following by trench support, foundation construction, pipeline installation and trench backfill. Due to the long distance of pipeline, the foundation treatment schemes vary from location to location. For the studied part of this project, light well point dewatering system, dumping rockfill and extruding silt approach are suggested.

Light well point dewatering system. Theoretically, when the width of trench is smaller than 6 m, single row light well point dewatering system can be presumed. For the case study, double row light well point dewatering system must be adopted since the 6 m criterion is violated. The pipeline cannot be immersed under the water, such that dewatering operation should last day and night.

Dumping rockfill and extruding silt method. The theoretical formulation is derived based on limit...
equilibrium theory with following assumptions: (a) The width of dumped rockfill is B, height is H, the deposition inside the mud is D and the height above mud surface is h; (b) The rockfill is presumed as elastic material with the unit weight of γ, where the frictional coefficient is neglected; (c) The mucky silty clay is considered as infinite half space, where the unit weight is γs, cohesion is c, and frictional angle is φ = 0. Therefore, the shear strength from cross plate shear test can be evaluated as τ+ = c; (d) Failure surface intersects with the principal stress plane by α = 45°. The schematic diagram of the calculation process can be illustrated as Fig. 4. Three zones are divided, where zone I, II and III present active earth pressure, transition region and passive earth pressure zone based on Rankine's theory.

The relation between rockfill height H and the deposition inside the mud D can be achieved by:

\[
H = \left(\frac{1}{2\pi} + 2\gamma s \cdot D \cdot \frac{1}{\gamma} \right) + \frac{(4\gamma s + 2\gamma D) \cdot D \cdot 2\gamma s \cdot D}{3\gamma \cdot B^2}.
\]

According to exploration survey report, the average shear strength of mucky silty clay is 7.9 kPa, the unit weight of mucky silty clay and rockfill is 17.59 kN/m³ and 28.65 kN/m³ respectively. The width of rockfill is assumed as 7 m. The calculation results from Equation (7) are summarized in Table 2.

![Figure 3. Schematic diagram of dumping rockfill and extruding silt method](image)

![Table 2. The calculation results of rockfill height, deposition inside the mud and height above mud surface](image)

<table>
<thead>
<tr>
<th>Deposition inside the mud /m</th>
<th>Rockfill height /m</th>
<th>Height above mud surface /m</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.48</td>
<td>1.39</td>
</tr>
<tr>
<td>2</td>
<td>4.91</td>
<td>2.76</td>
</tr>
<tr>
<td>3</td>
<td>6.84</td>
<td>3.93</td>
</tr>
<tr>
<td>4</td>
<td>8.92</td>
<td>5.07</td>
</tr>
<tr>
<td>5</td>
<td>11.36</td>
<td>6.48</td>
</tr>
<tr>
<td>6</td>
<td>14.16</td>
<td>8.12</td>
</tr>
<tr>
<td>7</td>
<td>17.03</td>
<td>10.18</td>
</tr>
<tr>
<td>8</td>
<td>20.17</td>
<td>12.28</td>
</tr>
</tbody>
</table>

5. NUMERICAL SIMULATION

The foundation treatment implementation of pipeline project was modelled using three dimensional finite differential method using continuum modeling tool FLAC3D (Bhardwaj 2006). The analysis modelled the mucky silty clay layer as elastic perfectly plastic hysteretic material. The cross section drawing of the trench excavation can be referred to Fig. 6. The length of numerical model parallel to the pipe axis, the width perpendicular to the pipe axis and height are taken as 40, 50.8 and 15 m respectively. As shown in Fig. 7, the top soil layer of silt is 1 m in height, below which, 1.4 m fine sand, 4.5 m mucky silty clay and 8.1 m medium-fine sand are modelled. Different colors are used to present...
different soil layers from top to bottom in blue, red, green and cyan respectively. The gravel bedding layer is colored in purple. All the boundary conditions are restrained in normal direction, and the top surface is free. The soil parameters for the case study are summarized in Table 3.

Table 3. Soil parameters for numerical model

<table>
<thead>
<tr>
<th>Soil layers</th>
<th>Width (m)</th>
<th>Density ρ (kg/m³)</th>
<th>Bulk modulus K (MPa)</th>
<th>Shear modulus G (MPa)</th>
<th>Cohesion c (kPa)</th>
<th>Frictional angle φ (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Silt</td>
<td>1</td>
<td>1810</td>
<td>9.25</td>
<td>3.78</td>
<td>6</td>
<td>22</td>
</tr>
<tr>
<td>Fine sand</td>
<td>1.4</td>
<td>1900</td>
<td>57.97</td>
<td>31.49</td>
<td>0</td>
<td>30.3</td>
</tr>
<tr>
<td>Mucky silty clay</td>
<td>4.5</td>
<td>1795</td>
<td>4.17</td>
<td>1.92</td>
<td>7.9</td>
<td>9.3</td>
</tr>
<tr>
<td>Medium-fine sand</td>
<td>8.1</td>
<td>1950</td>
<td>72.46</td>
<td>39.37</td>
<td>0</td>
<td>30.3</td>
</tr>
<tr>
<td>Gravel bedding</td>
<td>1.5</td>
<td>2200</td>
<td>92.59</td>
<td>40.65</td>
<td>0</td>
<td>34</td>
</tr>
</tbody>
</table>

Figure 5. Cross section of the trench

Parametric analysis has been done to optimize the foundation strengthening schemes. The rockfill height is assigned as five values of 1.00, 1.25, 1.50, 1.75 and 2.00 m respectively to investigate the load-settlement (p-s) curves of foundation bearing the pipes. The final decision of rockfill height was determined based on the numerical simulations. Five p-s curves are shown together in Fig.6.

Yielding point method can be used to determine the bearing capacity of foundation. As an interpretation of Fig.7, one can infer that the load is proportional to foundation settlement for the initial...
branch of p-s curves. The bearing capacity corresponding to the end of straight branch of p-s curves is denoted as allowable bearing capacity \( f_0 \). When the load increases until foundation failure, the ultimate bearing capacity is obtained as \( f_u \). The standard bearing capacity \( f_k \) can be expressed as follows:

\[
f_k = f_0 + \frac{f_u - f_0}{k}
\]

(9)

where, \( k \) varies due to soil type, shape and depth of foundation according to GB50007-2011 (2011), and is determined as 3 for the case study.

The calculated bearing capacity for five rockfill heights is summarized in Table 4.

<table>
<thead>
<tr>
<th>Rockfill height (m)</th>
<th>1.00</th>
<th>1.25</th>
<th>1.50</th>
<th>1.75</th>
<th>2.00</th>
</tr>
</thead>
<tbody>
<tr>
<td>Allowable bearing capacity (kPa)</td>
<td>80</td>
<td>100</td>
<td>130</td>
<td>150</td>
<td>170</td>
</tr>
<tr>
<td>Ultimate bearing capacity (kPa)</td>
<td>110</td>
<td>130</td>
<td>160</td>
<td>180</td>
<td>210</td>
</tr>
<tr>
<td>Standard bearing capacity (kPa)</td>
<td>90</td>
<td>110</td>
<td>140</td>
<td>160</td>
<td>183.3</td>
</tr>
</tbody>
</table>

According to GB50007-2011 (2011), the bearing capacity should reach 120 kPa after rockfill implementation. Table 4 demonstrates that when the rockfill height selects the value larger than 1.5 m, the bearing capacity can fulfill the requirements. Finally, 1.5 m rockfill height is suggested as the optimal solution.

**HEAVY DYNAMIC SOUNDING TEST**

Heavy dynamic sounding test is commonly used for field measurement. Conical probe falls down from a certain distance, where the variation of soil layer can be used to determine the soil resistance. The thump number N63.5 reflects the bearing capacity of foundation. The parameters used for this project are tabulated in Table 5.

<table>
<thead>
<tr>
<th>Diameter of conical bottom /m</th>
<th>Section area /cm²</th>
<th>Cone angle °</th>
</tr>
</thead>
<tbody>
<tr>
<td>74</td>
<td>43</td>
<td>60</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Hammer weight /kg</th>
<th>free-fall hammer distance /cm</th>
<th>Energy index J /cm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>63.5</td>
<td>76±2</td>
<td>115.2</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Probe diameter /mm</th>
<th>Penetration index</th>
<th>Penetration depth /m</th>
</tr>
</thead>
<tbody>
<tr>
<td>42</td>
<td>When penetration depth reaches 10cm, N63.5</td>
<td>12–16</td>
</tr>
</tbody>
</table>

Along the length of pipeline (stake number of 33+406~39+350), the 119 test points were set up for every 50 m. In order to testify the effectiveness of the dumping rockfill and extruding silt rehabilitation strategy, 12 test points for every 500 m were selected to compare with the numerical simulation results. The bearing capacity values measured from heavy dynamic sounding test are summarized in Table 6. Table 6 shows that the average bearing capacity from heavy dynamic sounding test is 148.4 kPa, which is larger than the design required value of 120 kPa. Compared with the bearing capacity obtained from numerical simulation, the field measurement has only 5.66% error, which also backward validates the correctness of numerical model. 1.5 m rockfill height is a reasonable and cost-effective solution for this pipeline construction project.

| Table 6. Bearing capacity obtained from heavy dynamic sounding test |
6. CONCLUSION
The stability of foundation is important for infrastructure (i.e. pipeline construction project). In this paper, second phase water supply project in Changxing Island Port Industrial Zone is used as case study to investigate the selection of rehabilitation strategy for pipes resting on the soft soil conditions. The treatment toward mucky silty clay by dumping rockfill and extruding silt was calculated through closed-form theoretical solutions using the parameters achieved from laboratory tests. FLAC3D numerical model was built to run parametric analysis to determine the rockfill height. Five rockfill height values were selected and load-settlement curves of foundation were available to check the bearing capacity. 1.5 m rockfill height was suggested and implemented to the pipe. The bearing capacity data obtained from heavy dynamic sounding test after completion of the project were used to compare with both numerical simulation and design code requirements. As an interpretation of the comparison results, one can infer that the numerical simulation supplies reasonable estimate of bearing capacity of foundation. The dumping rockfill and extruding silt with the rockfill height of 1.5 m is economic and effective.

REFERENCES
APPLICATION AND DEVELOPMENT OF RAISE BORING MACHINE IN PUMPED STORAGE POWER PLANT

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3. Beijing China Coal Mine Engineering Co., Ltd, 100013

Method of constructing vertical or large-inclined-angle pilot shaft was presented in this paper, constructing characteristics between Raise boring method, cage climbing method, and Suspended cage raising method was analyzed. Domestic raise boring technology and its application in construction of pumped storage power plant was discussed, some applied case were introduced.

Pumped storage power plant comprise of upper reservoir, lower reservoir, penstock, underground workshop and tailrace system. There are many vertical shafts and inclined shafts in these engineering. Among them, the penstocks and ventilation shafts include vertical and inclined shafts, but diversion and tailrace surge shaft is generally vertical. In construction of pumped storage power station with installed capacity of 1200MW, the total length of the vertical and inclined shafts is about 800-1500m. Although the engineering amount is small, but the construction is very difficult, especially in long-distance penstock construction it is the key to restrict the building of power plant. The lower exit generally existed in shaft constructions of pumped storage power plant. Pilot shaft is firstly drilled before the shaft isreamed to the design excavation section. Pilot shaft is the key engineering in the construction. In the traditional construction methods, workers are in insecurity, poor working conditions and great labor intensity in this condition, so the working efficiency is very low. This situation has seriously affected the pace of construction of pumped storage power plant; therefore, it must be changed.

1. Development of pilot shaft drilling technology

In the 1980s, ordinary Hydropower shaft construction often uses manual tunneling pilot shaft and manual raise boring method. Manual constructions have the characteristics of low speed and poor safety conditions. Although the construction of suspended cage raising method greatly increased the construction speed, but safety conditions are not improved. For example, in 1988, the pilot shaft of penstock was constructed by Alimarks raise climb method in the Guangzhou Pumped Storage Power plant; In 1991, we tried to construct pilot shaft by raise boring machine at Ming Tombs storage power plant, but failed due to deflection. In the power plant’s building, raising cage was introduced to the construction of penstock and surge tank. Raise cage method was not succeeded in Ming Tombs Due to the special geological conditions. Construction techniques of pilot shaft are shown in Table 1.

<table>
<thead>
<tr>
<th>item</th>
<th>Raise boring method</th>
<th>cage climbing method</th>
<th>Suspended cage raising method</th>
</tr>
</thead>
<tbody>
<tr>
<td>area</td>
<td>φ216～~3500mm</td>
<td>2m×2m or 2.4m×2.4m</td>
<td>φ1500mm</td>
</tr>
</tbody>
</table>

Table 1: Comparison of construction techniques
### 1.1 Manual pilot shafts construction

When the shaft is relatively shallow (generally less than 60 meters), manual boring construction method is used more often. Manual shafts construction includes ordinary construction method and raise boring construction method. The ordinary method is excavating pilot shafts from the top to the low level. As for shaft, derrick is installed at the head of shaft for staff and material transport. Workers enter into the underground workplace to drill, blast, clean up the waste rock and support. Mining method is often used in inclined shaft: the track is laid in the shaft; waste rock is hoisted with winch. The raise boring method is constructing the pilot shaft from the lower level. Then the platform is constructed. Workers come to the work face and complete construction of pilot shaft.

### 1.2 Suspended cage raising method

Suspended cage raising method can only apply in the construction of vertical shaft (Construction process shown in Figure 1). A pilot hole is bored to the downstairs in the center of the shaft. A hoist is installed on the upstairs. Wire rope which comes from the hoist is connecting and pulling up the raise cage through the drilling. At the beginning, the tools, equipment and workers are hanged up to the work place by the raise climb and the drilling work and charging begins. Then the raise climb is descended down to the bottom safety chamber, the personnel is withdrawn to a safe place to detonate. After blasting, ventilation and waterspray device is opened, the high-pressure air and water is sprayed out of the nozzles on the protection cover located on the top of the guideway, to exclude the blasting smoke. Rock debris is loaded in the underground roadway at the bottom of the shaft.

### 1.3 Raise Climb method

The technique of raise climb method is shown in Figure 2. First of all, blasting method is used to boring 3-5 meters above the underground roadway, and then installs the guide way and raise climb. At the beginning, the tools, equipment and workers are hanged up to the work place by the raise climb and the drilling work and charging begins. Then the raise climb is descended down to the bottom safety chamber, the personnel is withdrawn to a safe place to detonate.
shaft and workers can drill on the work place at the same time. When the pilot shaft is bored up to the upstairs, it needs to blast from the upstairs to complete the whole excavation process. Finally, the guide way can be removed top to down after work finish.

Compare with suspended cage raising method, raise climb method has the similar advantage of efficiency, speed and labor intensity. Its drawback is costly in equipment, hard to ventilate and easy to gather harmful gases on the work place. With the increasing depth of construction, efficiency slowed down as far as auxiliary time lengthened. Suspended bulkhead raising method needs not a tunnel upstairs. So this method can be used to construct the shaft of 1000 meters in theory. However, due to restriction by exhaust emission, this method is usually used for the shaft which depth not more than 150m.

1.4 Pilot shaft drilled by raise boring machine

Raise boring machine is the mechanical equipment which can work continuously. First the RBM is mounted on upper part of the concrete basis, then a guide hole is bored reach to the underground roadway in the center of the shaft (As shown in figure 3). After the guide hole has been completed, the guide drill bit is taken down and the reaming drill bit is connected. Then reaming is begun upward. The cuttings in drilling guide hole are taken out of the ground by liquid circulation and the cuttings in reaming are fallen down by gravity which is shipped out by the loading equipment.

2. Development of RBM technology
Because of the complex conditions of the coal mine, the raise boring machine was developed for the construction of the winze, coal bunker and ventilation shaft. The key National Seventh Five-Year Plan Research and Development Project-Design and Manufacture of the LM-120 series were undertaken by the shaft construction branch of China Coal Research Institute. The manufacture machine was completed in July 1986. In May 1987, industrial test of the machine was completed and passed by the identification of the Former Ministry of Coal. In 1987 to 1989, LM-200 series was developed and LM-90 was developed in 1992. Then the ZFY2.0/400 (BMC400) was prepared in 2006. By 2007~2008, new series of RBM which called ZFY5.0/600 (BMC600) was developed to construct shafts with diameter of 5m and depth of 600m.

The LM series were developed in 1980s and not suited for deep shafts and hard rock in hydropower construction ever. So the BMC series were developed to solve this problem. BMC300 (ZFY1.4/300) is alternative to LM-200 which is universal application of hydropower construction. Relative to the LM200, BMC300 get performance improvement, but it still retains the characteristics of small size, light weight and easy to move and apply. Other large-scale RBM would play a greater role in the construction of pumped-storage power plant. Main technical parameters of BMC series are shown in Table 2.

<table>
<thead>
<tr>
<th>technical parameter</th>
<th>BMC100 (ZFY1.0/100)</th>
<th>BMC200 (ZFY1.2/200)</th>
<th>BMC300 (ZFY1.4/300)</th>
<th>BMC400 (ZFY2.0/400)</th>
<th>BMC500 (ZFY3.5/500)</th>
<th>BMC600 (ZFY5.0/600)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Name</td>
<td>unit</td>
<td>unit</td>
<td>unit</td>
<td>unit</td>
<td>unit</td>
<td>unit</td>
</tr>
<tr>
<td>Diameter of reaming</td>
<td>m</td>
<td>1.0</td>
<td>1.2</td>
<td>1.4</td>
<td>2.0</td>
<td>2.0~3.5</td>
</tr>
<tr>
<td>Diameter of pilot hole</td>
<td>mm</td>
<td>216</td>
<td>216</td>
<td>244</td>
<td>270</td>
<td>311</td>
</tr>
<tr>
<td>Depth of reaming</td>
<td>m</td>
<td>100</td>
<td>200</td>
<td>300</td>
<td>400</td>
<td>500</td>
</tr>
<tr>
<td>Diameter of rig</td>
<td>mm</td>
<td>176</td>
<td>182</td>
<td>203</td>
<td>228</td>
<td>254</td>
</tr>
<tr>
<td>Thrust</td>
<td>kN</td>
<td>200</td>
<td>350</td>
<td>550</td>
<td>1650</td>
<td>1500</td>
</tr>
<tr>
<td>Tension</td>
<td>kN</td>
<td>500</td>
<td>850</td>
<td>1250</td>
<td>2450</td>
<td>3000</td>
</tr>
<tr>
<td>Torque</td>
<td>kN.m</td>
<td>20</td>
<td>35</td>
<td>64</td>
<td>80</td>
<td>120</td>
</tr>
<tr>
<td>Main engine power</td>
<td>kW</td>
<td>62.5</td>
<td>86</td>
<td>128.5</td>
<td>128.5</td>
<td>172.5</td>
</tr>
<tr>
<td>Rotating speed</td>
<td>rpm</td>
<td>15~20</td>
<td>20~30</td>
<td>20~30</td>
<td>20~30</td>
<td>15000</td>
</tr>
<tr>
<td>Inclination</td>
<td>°</td>
<td>60~90</td>
<td>60~90</td>
<td>60~90</td>
<td>60~90</td>
<td>Electrohydraulic</td>
</tr>
<tr>
<td>Size in work</td>
<td>m</td>
<td>2.4×1.0×1.27×2.92</td>
<td>2.9×1.4×3.25</td>
<td>3.5×1.75×3.48</td>
<td>4.85×1.9×5.25</td>
<td>4.85×1.9×5.5</td>
</tr>
<tr>
<td>Size for transportation</td>
<td>m</td>
<td>2.4×1.0×1.065</td>
<td>2.9×1.1×1.67</td>
<td>3.0×1.4×1.7</td>
<td>3.5×1.4×1.75</td>
<td>Removable</td>
</tr>
<tr>
<td>Weight of host</td>
<td>kg</td>
<td>3.5</td>
<td>7.9</td>
<td>8.7</td>
<td>12.5</td>
<td>18</td>
</tr>
<tr>
<td>Driving mode</td>
<td></td>
<td>Hydraulic</td>
<td>Hydraulic&amp;Hydraulic control</td>
<td>Hydraulic&amp;Hydraulic control</td>
<td>Hydraulic/Electrohydraulic</td>
<td>Hydraulic/Electrohydraulic</td>
</tr>
</tbody>
</table>
3. Application of raising boring machine in constructing Pumped-storage Power Plant

3.1 Ming Tombs Pumped-storage Power Plant

Ming Tombs Pumped-storage Power Plant has 11 shafts, including NO.1 and NO.2 diversion emergency gate shafts, NO.1 and NO.2 the intake Surge Shafts, the outgoing shaft, tailrace emergency gate shafts from NO.1 to NO.4 with NO.1 and NO.2 tailrace surge shafts. The total deep of shafts are 552 meters, which a single depth ranging from 16 m to 160.8 m. NO.1 and NO.2 Penstock consists of five sections including flat region, acclivity region, central flat region, descent region and the lower region upper flat region, upper flat region, central flat region, lower inclined region, lower flat region. The length of inclined region is 1197.88 m and the horizontal angle is 50°. The shafts are located in the andesite and conglomerate. Most of surrounding rock classification is III ~ IV. By the influence of faults and joints, poor geological and rich groundwater conditions which cause great difficulties to the construction of shafts. The construction of four shafts are completed by RBM, the power plant’s construction is sped up (table 3).

<table>
<thead>
<tr>
<th>Name of shafts</th>
<th>Surrounding Rock Classification</th>
<th>Length /m</th>
<th>Angle /°</th>
<th>Diameter /m</th>
<th>RBM Model</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>intake surge shafts</td>
<td>III-IV</td>
<td>50</td>
<td>90</td>
<td>φ0.9</td>
<td>LM-90</td>
<td></td>
</tr>
<tr>
<td>outgoing shaft</td>
<td>II - III</td>
<td>160.8</td>
<td>90</td>
<td>φ1.4</td>
<td>LM-200</td>
<td>first time for RBM constructing the power plant</td>
</tr>
<tr>
<td>NO.1 descent region</td>
<td>III</td>
<td>237</td>
<td>50</td>
<td>φ1.4</td>
<td>LM-200</td>
<td>success</td>
</tr>
<tr>
<td>NO.2 descent region</td>
<td>II - IV</td>
<td>193</td>
<td>50</td>
<td>φ1.4</td>
<td>LM-200</td>
<td>first time for RBM constructing Inclined shaft</td>
</tr>
</tbody>
</table>

3.2 Mount Langya pumped-storage power plant

Raising boring machine was used in the construction of outgoing shaft, smoke exhaust shaft, and NO.1 to NO.4 penstock of Mount Langya pumped-storage power plant. According to project, new tailrace tunnel and ventilation shaft was added. The first LM-120 series was equipped in October 31, 2002. As of October 23, 2003, the seven shafts were completed, including outgoing shaft, smoke exhaust shaft, penstock and so on. The total depths of shafts were 959.35 m. Another LM-400 series RBM was added in the construction of smoke exhaust shaft of the workshop. All guide shafts’ construction and technological statistics are shown in table 4.

<table>
<thead>
<tr>
<th>Number</th>
<th>Name</th>
<th>Diameter/m</th>
<th>Depth/m</th>
<th>Type of RBM</th>
<th>Deviation rate</th>
<th>Pilot hole/d</th>
<th>Reaming period/d</th>
<th>Illustrate</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>outgoing shaft</td>
<td>1.4</td>
<td>134.55</td>
<td>LM-120</td>
<td>0.59%</td>
<td>7</td>
<td>8</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Tailrace exhaust</td>
<td>1.4</td>
<td>142</td>
<td>LM-120</td>
<td>0.6%</td>
<td>10</td>
<td>11</td>
<td>Passing</td>
</tr>
</tbody>
</table>
### 3.3 Pushihe Pumped storage power plant

The effective capacity of upper reservoir of Pushihe Pumped storage power plant is 10.29 million m³ and lower reservoir is 1284 m³, with 1200MW installed capacity. The power station needed to construct the shaft and inclined shaft, including main workshop ventilation shaft, auxiliary workshop ventilation shaft, tailrace gate ventilation shaft, surge shaft and penstock. To speed up the construction, a mid-construction adit was set in the middle of the penstock. So the penstock was divided into two parts, in order to facilitate the construction of pilot hole, the construction of reaming and pipe installation. Surrounding rock belongs to the class I - II, and relatively stable. Within 10m of the main and auxiliary ventilation shafture strong or weak weathered beds, the other rock is composition of hybrid granite, including granite, granitic gneiss and basalt. Rock uniaxial compressive strength is 15 — 25MPa. Strata confined water does not exist. But in the rainy season, there was seepage in some place. Seepage is less than 5m³/h.

Table 5 construction and technical statistics of pilot shaft

<table>
<thead>
<tr>
<th>Number</th>
<th>Name</th>
<th>Horizontal angle $\phi$</th>
<th>Depth /m</th>
<th>Diameter/m</th>
<th>Commencement date</th>
<th>Completion date</th>
<th>Deviation rate</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>ventilation shaft for auxiliary workshop</td>
<td>90</td>
<td>213.88</td>
<td>1.4</td>
<td>2007.04.02</td>
<td>2007.0504</td>
<td>0.41%</td>
</tr>
<tr>
<td>2</td>
<td>ventilation shaft for main workshop</td>
<td>90</td>
<td>258.20</td>
<td>1.4</td>
<td>2007.08.22</td>
<td>2007.11.06</td>
<td>0.50%</td>
</tr>
<tr>
<td>3</td>
<td>ventilation shaft for tailrace gate</td>
<td>90</td>
<td>22.50</td>
<td>1.2</td>
<td>2007.10.01</td>
<td>2007.10.09</td>
<td>0.8%</td>
</tr>
<tr>
<td>4</td>
<td>surge shaft</td>
<td>90</td>
<td>111.00</td>
<td>1.4</td>
<td>2007.12.13</td>
<td>2007.12.31</td>
<td>0.35%</td>
</tr>
<tr>
<td>5</td>
<td>2# lower inclined shaft</td>
<td>55</td>
<td>218.09</td>
<td>1.4</td>
<td>2007.12.08</td>
<td>2008.01.30</td>
<td>0.90%</td>
</tr>
<tr>
<td>6</td>
<td>1# upper inclined shaft</td>
<td>54</td>
<td>170.07</td>
<td>1.4</td>
<td>2008.02.22</td>
<td>2008.04.01</td>
<td>0.8%</td>
</tr>
<tr>
<td>7</td>
<td>2# upper inclined shaft</td>
<td>55°39’</td>
<td>161.41</td>
<td>1.4</td>
<td>2008.04.17</td>
<td>2008.05.22</td>
<td>0.83%</td>
</tr>
<tr>
<td>8</td>
<td>1# lower inclined shaft</td>
<td>54°39’</td>
<td>210.48</td>
<td>1.4</td>
<td>2008.04.30</td>
<td>2008.06.21</td>
<td>0.75%</td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td></td>
<td>1365.63</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
The first BMC300 type raise-boring machine equipping from April 2007, after it completed the main and auxiliary ventilation shaft, the second BMC300 series entered the construction, completing a surge shaft and four penstocks, two raise-boring machines together to complete the shaft 605.58m and inclined shaft 760.05m, total 1365.63m, the construction site as shown in Figure 4, 5. #1, 2# diversion inclined is the most difficult segment of the main project construction works, the length of 1# diversion inclined is 380.55m, diameter is 9.3m, inclination angle is 54°46′11″, the length of 2# diversion inclined is 379.55m, diameter is 9.3m, inclination angle is 55°0′00″, the horizontal spacing of the two inclined is about 45m, divided into four sections to dig, the four inclined shafts use BMC300 type raise-boring machine to drill guidance shaft excavation and construction, technical statistics is shown in table 5 (fig 4).

4. Conclusion

(1) Raise-boring machine is applied from mining to other underground projects, especially in the Pumped Storage project, and is played a huge role in engineering construction, which makes an important significance for the promotion of raise-boring machine.

(2) The raise-boring technology is applying from the Ming Tombs Pumped-storage Power Plant outgoing shaft, which has obvious advantages compared with the raise climb method, suspended cage raising method and manual raise boring method, by which the staff operated in a safe place, entered into the dangerous mining workplace no longer, the operating environment is good and the labor intensity got improving. Later, we make an attempt of raise-boring machine to the construction of large-angle inclined shaft, and a inclined pilot shaft of pressure pipe in NO.2 penstock which depth is 203m and inclination 50 degrees was completed. The success of these projects has opened up new application for raise-boring machine. Since then, raise-boring machine has been widely used in power plant construction, becoming major equipment in construction of the pilot shaft.
TEST RESEARCH ON FAILURE MECHANISM OF ROCK-SOCKETED PILE IN KARST AREA

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Based on indoor model test, the research on failure mechanism of rock-socketed pile was carried on. The test indicated that, the failure process was divided into three stages, they were stably bear load stage, cracks occurred stage and failure stage, there were three special zones, they were tensile area near pile, compression area under pile tip and tensile area under a certain depth, the failure reason maybe tensile shear failure or compression shear failure, and cave roof thickness 3d was the critical value of different failure modes. Known from test results, the failure modes could be divided into four types, they were punching failure, collapsed zone failure, sector plastic zone failure and tearing failure.

1 Introduction

In Karst Area, rock-socketed piles are widely used in various construction engineering. As a result of karst development, especially the cave existence, a lot of troubles and problems were brought to construction engineering, so it need people to put into in-depth study on bearing capacity.

In this article, based on model test, the influence to rock-socketed pile’s failure mechanism and failure mode in Karst area with different influencing factors were researched, such as roof thickness, cave migration position, cave equatorial and polar radius.

2 Preliminary Study on Failure Mechanism

The foundation failures were divided into three modes by soil mechanics which are general shear failure, local shear failure and punching failure. The failure process generally experienced three stage, these are compression phase, shearing phase and failure stage. The foundation failure mode related to soil compressibility, depth of foundation and loading rate, etc. The rock failures were divided into three modes by rock mechanics which are brittle failure, ductile failure and weak surface shear failure. When determined the rock foundation’s ultimate bearing capacity, a certain plastic equilibrium zone in rock foundation was assumed, this moment the foundation will slide along a continuous sliding surface. Discuss on the stability of cave surrounding rock, related theory are pressure arch theory, Terzaghi theory and elastic-plastic theory.

Combined with soil mechanics and rock mechanics related theory, it was suggested that the failure of rock-socketed pile in Karst area was a gradual change process. The failure pattern was a transfixion continuous sliding surface, and the influencing factors to concrete failure mode except for deformation modulus and buried depth etc., also related with cave and joint fissure etc. Because of the concrete materials of rock-socketed piles were always C25-40 which elastic modulus were 28-32.5GPa, and the rock foundations are also moderately weathered limestone which elastic modulus were 21-84GPa, the
occurred possibility of punching failure was very small. As for general shear failure of shallow foundation, because the overlying soil was rather thick, the occurred possibility was also very small.

3 Brief Description of Model Test

The geometric similarity ratio was 1/20, the pile length of archetype rock-soketed pile was 10.8m, pile diameter was 0.8m, buried depth in rock was 0.8m, and the interaction system was considered as central symmetric model. In model test, the physico-mechanical parameters and model material were seen in table 1.

<table>
<thead>
<tr>
<th>Tab. 1 Physico-mechanical parameters and model material</th>
</tr>
</thead>
<tbody>
<tr>
<td>Material types</td>
</tr>
<tr>
<td>Bulk density $\gamma$(kN/m$^3$)</td>
</tr>
<tr>
<td>Compressive Strength $\sigma_c$(Mpa)</td>
</tr>
<tr>
<td>Tensile strength $\sigma_t$(Mpa)</td>
</tr>
<tr>
<td>Elastic modulus $E$(Gpa)</td>
</tr>
<tr>
<td>Poisson ratio $\mu$</td>
</tr>
<tr>
<td>$C$(MPa)</td>
</tr>
<tr>
<td>$\phi$(°)</td>
</tr>
</tbody>
</table>

Five different influencing factors were considered in this scheme (see table 2).

<table>
<thead>
<tr>
<th>Tab. 2 Scheme of model test</th>
</tr>
</thead>
<tbody>
<tr>
<td>Change factors</td>
</tr>
<tr>
<td>Roof thickness $H$</td>
</tr>
<tr>
<td>Cave diameter $D$</td>
</tr>
<tr>
<td>Cave position $l$</td>
</tr>
<tr>
<td>Cave polar radius $c$</td>
</tr>
<tr>
<td>Cave equatorial radius $a$</td>
</tr>
</tbody>
</table>

4 Model Test on Failure Mechanism

There are two methods to study on failure mechanism of rock-sOCKETED pile in Karst area, these are direct observation (see Fig.1a) and inner strain measuring technology (see Fig. 1b).

![Fig. 1 Test system of central symmetric model](image)

4.1 Observation of Test Process

The entire test process was recorded by high precision photography equipment. As for the block having obvious damage zone, cave roof will occurred sudden failure, and the failure process was that rock foundation stably bear load, then the cracks were occurred, finally, with the increasing of load, the
cracks rapidly extended and the cave roof was collapse. As for the block no having obvious damage zone, cave roof was no change in basic, the plastic equilibrium zone was formed above the cave.

Take cave diameter $3d$ and roof thickness $1d$ as the example. When load $<3.0kN$, the rock foundation can stably bear load, and the cracks was not occurred in cave roof (see Fig. 2a). When load increased to 3.0kN, the cracks was occurred and expanded near pile tip, the cracks just like eight (see Fig. 2b). When load increased to 3.5kN, the block collapse like frustum was occurred (see Fig. 2c).

![Fig. 2 Photos of failure process (H=1d, D=3d)]

4.2 Strain Measurement of Test Process

(1) Cave diameter $3d$ and roof thickness $3d$

Take cave diameter $3d$ and roof thickness $3d$ as the example, the strain contour maps with different load classes could be seen in Fig. 3.

![Fig. 3 Strain contour maps with different load classes (H=3d, D=3d)]

(2) Cave diameter $5d$ and roof thickness $2d$
Take cave diameter $5d$ and roof thickness $2d$ as the example, the strain contour maps with maximum load could be seen in Fig. 4.

(3) Roof thickness $2d$ and polar radius $1d$

Take roof thickness $2d$ and polar radius $1d$ as the example, the strain contour maps with maximum load could be seen in Fig. 5.

4.3 Analysis on Failure Mechanism

(1) Analysis on Failure Process

Known from Fig. 2-5, the concentration phenomenon of maximum principal strain near pile tip was occurred, and the strain was compressive strain. When load was small, the minimum principal strain near pile side was negative, and the tensile area was existed. With load increasing, the minimum strain could be divided into 3 areas, they were tensile area near pile, compression area under pile tip and tensile area under a certain depth, the concentration phenomenon of compression and two tensile areas was occurred with load increasing.

Combined with Fig. 2, the cracks were firstly occurred near pile tip, the cracks were controlled by tensile strength of similar material, and the failure mode was tensile shear failure.

Assumption that the simulation materials were elastic-plastic body, the failure mode obeyed Mohr-Coulomb strength criterion. Take cave diameter $3d$ and roof thickness $3d$ as the example, the compressive strain and tensile strain when failure occurred were respectively for $\varepsilon_c = \sigma_c / E = 2.0 / 710 \times 10^6 = 2816.9$ and $\varepsilon_t = \sigma_t / E = -0.29 / 710 \times 10^6 = -408.5$. When shear failure was occurred, the relationship of maximum strain $\varepsilon_1$ and minimum strain $\varepsilon_3$ was $\varepsilon_1 - 3.0\varepsilon_3 = 2563.4$. 

Considered the influence of under glue and adhesive, testing errors of strain rosette were existed. Combined with Fig. 3, the maximum strains $\varepsilon_1$ were less than 2816.9, so the compression-destroying of similar rock could not be occurred. When ultimate load, the minimum strain $\varepsilon_3$ of a certain depth tensile area under pile tip was about -400, it’s close to ultimate tensile strain, the maximum value of maximum strain $\varepsilon_1$ was about 800, while $\varepsilon_1-3.0\varepsilon_3=2000$ which approached 2563.4, the minimum strain $\varepsilon_3$ of tensile area near pile side was about 350, while $\varepsilon_1-3.0\varepsilon_3=800$ which had large gaps to critical value of failure mode. So the failure was firstly occurred at the top of the certain depth tensile area, then expanded and formed failure zone, the failure reason maybe tensile shear failure or compression shear failure.

Known from Fig. 4 and Fig. 5, the maximum strains $\varepsilon_1$ were less than 2816.9, so the compression-destroying of similar rock could not be occurred. When ultimate load, the minimum strain $\varepsilon_3$ of a certain depth tensile area under pile tip was bigger than -400, so the failure mode was tensile shear failure.

So, the failure process could be divided into three stages, and cave roof thickness $3d$ was the critical value of different failure modes.

(2) Summarize of Failure Mode

The failure types of different influence factors could be seen in Fig. 6-10.

Fig. 6 Failure modes with different roof thicknesses

Fig. 7 Failure modes with different cave diameters

Fig. 8 Failure modes with different polar radius
From the model test results, the failure modes of rock-socketed pile in Karst area could be divided into four types, they were punching failure, collapsed zone failure, sector plastic zone failure and tearing failure. When the roof thickness $H$ was thin, the failure mode was punching failure (see fig. 11a), and failure was tensile shear failure. When the roof thickness $1d < H < 3d$, the failure mode was collapsed zone failure (see fig. 11b). When the roof thickness $H \geq 3d$, the failure mode was sector plastic zone failure (see fig. 11c). When the rock foundation had many vertical fractures liking elliptic, the failure mode was tearing failure (see fig. 11d).

5 Conclusions

In this paper we can get these conclusions.

(1) The failure process was divided into three stages, they were stably bear load stage, cracks occurred stage and failure stage.

(2) There were three special zones, they were tensile area near pile, compression area under pile tip and tensile area under a certain depth, the failure reason maybe tensile shear failure or compression shear failure, and cave roof thickness $3d$ was the critical value of different failure modes.
(3) The failure modes could be divided into four types which were punching failure, collapsed zone failure, sector plastic zone failure and tearing failure

References


MODEL TESTS ON MECHANICAL BEHAVIOR OF UNDERREAMED GROUND ANCHOR

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Based on the research of new anchor technique, anti-corrosive capsule underreamed ground anchor was developed. After 25 model tests of anchors with underreamed parts conducted, the definition and disparity of shallow and deep underreamed ground anchors were put forward. The results also elaborated the differences between shallow and deep anchors in failure modes and bearing behavior. And the Q-S curves and the balloon failure shape of deep underreamed ground anchor indicated that the Q-S curves of deep anchor showed a monotonic increase in mechanical properties due to the contribution of the underreamed part. Thereby, deep underreamed ground anchor had higher bearing capacity and better safety with high value in geotechnical market.

1 Introduction

With the rapid development of national economy and infrastructure in China, a large number of giant transport hubs, multi-storey underground shopping malls, projects of settling ponds and underground civil air defense projects emerge in large and medium-sized cities. Active anti-floating technology, such as drainage decompression and the technology of passive anti-floating, are all used for anti-floating stability. Among passive anti-floating technologies, the anti-floating anchor, as one of the core technologies of geotechnical anchoring, with its obvious technical advantage and cost effective, has gained increasing attention in the civil engineering sector.

In order to improve the bearing capacity, safety and durability of the ground anchor to cope with the complexity of geological conditions and the diversity of engineering requirements, since the 1970s, underreamed anchors by machinery, explosions and hydraulic means began to appear in construction fields. Britain, France, Sweden, Czech Republic, Japan, as well as China Taiwan has developed underreamed anchors respectively.

Based on previous research and practice, anti-corrosive capsule underreamed ground anchor (CJY-KT anchor) has been developed by China Jingye Engineering Corporation. And series model tests had been carried out to obtain the failure modes and bearing mechanism of the new anti-floating anchor. Up to now, this new technology has been successfully applied to actual projects and gained numbers of national patents in China.
2  Failure modes of underreamed ground anchor for anti-floating

In order to study the mechanical properties of underreamed ground anchor under controlled conditions, small-scale model tests were carried out, focusing on the failure modes and the bearing mechanism of the new type of anchor.

2.1 Equipment and programs of model tests

20 layers sand box was used in the model tests, and the whole size of the box was 0.8m(length)×0.7m(weigh)×1.2m(H). The simulated foundation was prepared through stratified sand raining with medium quartz sand. The physical and mechanical properties of the simulated foundation were shown in Table 1. The models of underreamed anchor were made of steel cylinders with shallow screw threads of different sizes and connected with screw bar of 8mm diameter.

| Table 1 Physico-mechanical parameters of model sand. |
|-----------------|-----------------|-----------------|-----------------|-----------------|
| Density /g·cm-3 | Coefficient of uniformity | Dry density /g·cm-3 | Specific gravity | Max dry density /g·cm-3 |
| 1.49            | 1.9              | 1.49            | 2.67            | 1.60            |
| Relative density | Cohesion /kN·m-2 | Friction angle /° | Poisson's ratio/° | Min dry density /g·cm3 |
| 0.673           | 0.0              | 40.0            | 0.26            | 1.30            |

Model test system includes fine-controlled loading frame (Figure 1), pulling system (Figure 1), load force measurement device, deformation measurement system, and close-range digital photography deformation measurement system as well. The programs of failure modes and bearing mechanism of underreamed ground anchor were respectively listed in Table 2 and Table 3. The embedment ratio of underreamed ground anchor was defined. The embedment ratio (T/D) was the specific value between T (the distance between the top surface of the underreamed part and the surface of the soil) and D (the diameter of the underreamed part). Moreover, the meanings of L and H are the length of the underreamed part and the total length of the anchor respectively.

2.2 Model tests of failure modes and analysis of results

In order to study the failure modes of underreamed anchor, scholars researched a large number of plate anchors with model tests all over the world, and many different failure modes were assumed. By assuming various shapes of failure surface, different methods were proposed in the past to predict...
bearing capacity of anchors. The friction cylinder method was one of the earliest methods adopted for the design of anchors (Majer, 1955). Mors (1959) assumed a truncated cone extending above the base of the anchor with an apex angle. Mariupol’skii (1965) also found that the shallow and deep anchor behaviour are different, he described the rupture surface as a tunnel through the soil (punching shear). Matsuo (1967, 1968) approximated the rupture surface to be composed of a logarithmic spiral and a tangential plane surface. Murray and Geddes (1987) considered a curved failure surface in their limit equilibrium analysis for the estimation of pullout resistance of anchors. However, the curvature was not defined. Besides, Ghaly (1991) put forward that there are different failure modes between shallow and deep underreamed ground anchor based on model tests of screw anchors. And S.T.Hsu and H.J.Liao (1998) accounted underreamed ground anchor could be divided into shallow and deep forms depending on the difference of embedment ratio based on cylindrical underreamed anchor.

Based on all above research methods and results of model tests, special test and measurement methods were adopted to get true failure mode of underreamed ground anchor, in which the accurate vertical pulling force and the deformation of soil around anchors were observed. And a special close-range digital photography deformation measurement method was put into use.

Table 2 Model test programme of failure modes of underreamed ground anchors.

<table>
<thead>
<tr>
<th>Program</th>
<th>Length of underreamed part L/mm</th>
<th>Diameter of underreamed part D/mm</th>
<th>Embedment depth T/mm</th>
<th>Embedment ratio T/D</th>
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</thead>
<tbody>
<tr>
<td>Shallow</td>
<td>100</td>
<td>80</td>
<td>400</td>
<td>5</td>
</tr>
<tr>
<td>Deep</td>
<td>100</td>
<td>80</td>
<td>850</td>
<td>10.63</td>
</tr>
<tr>
<td>Deep</td>
<td>100</td>
<td>60</td>
<td>850</td>
<td>14.17</td>
</tr>
</tbody>
</table>

Table 3 Model test programme of bearing capacity of underreamed ground anchors.

<table>
<thead>
<tr>
<th>No.</th>
<th>Length of underreamed part L(mm)</th>
<th>Diameter of underreamed part D(mm)</th>
<th>Embedment depth T(mm)</th>
<th>Embedment ratio T/D</th>
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</thead>
<tbody>
<tr>
<td>1</td>
<td>100</td>
<td>100</td>
<td>1050</td>
<td>10.5</td>
</tr>
<tr>
<td>2</td>
<td>100</td>
<td>80</td>
<td>1050</td>
<td>13.1</td>
</tr>
<tr>
<td>3</td>
<td>100</td>
<td>60</td>
<td>1050</td>
<td>17.5</td>
</tr>
<tr>
<td>4</td>
<td>100</td>
<td>40</td>
<td>1050</td>
<td>26.3</td>
</tr>
<tr>
<td>5</td>
<td>100</td>
<td>20</td>
<td>1050</td>
<td>52.5</td>
</tr>
<tr>
<td>6</td>
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<td>950</td>
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<td>7</td>
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<td>950</td>
<td>11.9</td>
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<td>100</td>
<td>60</td>
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<td>15.8</td>
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<tr>
<td>9</td>
<td>100</td>
<td>40</td>
<td>950</td>
<td>23.8</td>
</tr>
<tr>
<td>10</td>
<td>100</td>
<td>20</td>
<td>950</td>
<td>47.5</td>
</tr>
<tr>
<td>11</td>
<td>100</td>
<td>100</td>
<td>850</td>
<td>8.5</td>
</tr>
</tbody>
</table>

The typical failure mode of shallow anchor (T/D=5, L=100mm, D=80mm, T=400mm) is shown in figure 2(a).
As the figure shown, the vertical displacements of soil particles on all horizontal planes above the underreamed part decreased in the radial directions from the center and the decrease was rapid towards the rupture surface. There are also radial displacements of the soil particles away from the center of the anchor. The resultant displacement was inclined away from the axis of the anchor, except on the axis itself, where it was vertical. The inclination increased away from the axis of the anchor, towards the rupture surface.

The rupture surface emerged from the edge of the anchor and enlarged radially towards the top, convex upwards, with a very gentle curvature. It can be approximated to a plane surface which makes an angle of 40° to the horizontal. It is almost the friction angle of the sand. The top of the invert bell-shaped rupture surface exhibited significant vertical deformation when the pulling displacement of the anchor was 60mm. Therefore the whole rupture surface showed general shear failure characteristics. As this destructive phenomenon will lead to unexpected consequences of disaster, therefore, this shallow type of underreamed ground anchor must be forbidden in engineering design.

The rupture surface obtained for the anchor with underreamed part of 80 mm diameter and 100mm length embedded at 850 mm depth in sand bed is presented in Figure 2(b). The shape of the rupture surface was closed balloon in the case of 60mm displacement of the anchor.

The rupture surface also emerged from the edge of the underreamed part. The whole rupture surface was confined to a certain height above the anchor and they are smaller than the anchor displacement. The plane rupture surface made an angle of 43° to the horizontal, which was about 1.08 times the friction angle value of the sand.

The main failure characteristics of the deep underreamed anchor: the failure surface was enclosed within the overlying soil, therefore, the failure mode was local shear failure. In essence, with the continuous increase of vertical displacement, the balloon-shaped rupture surface was constantly larger, the outer surface area was increasing, the surrounding soil was also compressed. Thus as long as there still retained a certain thickness of the undeformed soil layer, the underreamed ground anchor would show a continuous slight increase.
## 3 Bearing characteristics of anti-floating underreamed ground anchor

Based on the analysis results of failure modes of underreamed ground anchor, according to the programs of Table 3, the model test of bearing deformation characteristics was carried out. 22 Q-S curves under different conditions were obtained by varying the depth, the embedment ratio and the size of underreamed part. Due to space limited, only part of the measured Q-S test curves had been analyzed, shown in Figure 3(a).

### Table 4 Ultimate bearing capacity and corresponding displacement of anchors in model test

<table>
<thead>
<tr>
<th>No.</th>
<th>Length L/mm</th>
<th>Diameter D/mm</th>
<th>Embedment ratio T/D</th>
<th>Ultimate bearing capacity Qu/N</th>
<th>Corresponding displacement Su/mm</th>
</tr>
</thead>
<tbody>
<tr>
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<td>100</td>
<td>100</td>
<td>10.5</td>
<td>2830</td>
<td>28.6</td>
</tr>
<tr>
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<td>100</td>
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<td>13.1</td>
<td>2390</td>
<td>28.3</td>
</tr>
<tr>
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<td>100</td>
<td>60</td>
<td>17.5</td>
<td>1550</td>
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</tr>
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<td>26.3</td>
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</tr>
<tr>
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<td>100</td>
<td>9.5</td>
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</tbody>
</table>

In Figure 3(a), the load versus displacement curves for anchors with underreamed parts of 60 mm, 80 mm, and 100 mm diameter with depth of embedment of 500 mm, 650 mm, and 850 mm are presented. The three curves indicated that for shallow underreamed anchor, the load increased rapidly with displacement and reached a peak, after that, the load rapidly decreased and then remained more or less constant or with a gentle decrease on further pulling. The following characteristics can be summarized from measured Q-S test curves of shallow anchors.

1) At the beginning of loading, there was a rapid increase in the load with displacement, at this stage, the anchor and soil demonstrated linear elastic deformation.

2) After the first turning point, the curve gradually developing to the peak of the Q-S curve remained linear development, but the slope of the straight line segment decreased, and the displacement rate increased, the anchor and soil were still in elastic stage.

3) When reaching the peak point, the curve began to decrease, and finally reached a stable value, that is, with the displacement gradually increasing, the load changed very little. After peak point, anchor and soil system went into plastic stage, and eventual general shear failure took place.
Figure 3(b) shows the typical shape of measured load versus displacement curves for deep anchors of T/D=9.5~11.25. Different from shallow anchor, peak point and decrease stage did not appear in the Q-S curves of deep underreamed anchors. At the latter stage of loading, a small number of curves performed stably, while most curves performed monotonically increasing.

Figure 3(b) demonstrated the basic bearing and deforming characteristics of deep underreamed anchor, that is, the displacement ratio of the Q-S curve of deep anchor is low at the beginning of loading, and then it got greater. The whole curve shows a monotonic increase in features. Setting forth in the previous section, the failure surface of deep anchor was a closed balloon, it can be inferred that the failure balloon would gradually extend with the pulling load increasing. On the contrary, the increasing load would spread to larger confined area with the failure balloon extending. As long as there was sufficient thickness of the overlaying soil, and the pulling displacement could meet the service conditions of buildings, the underreamed ground anchor could possibly obtain higher end bearing capacity, so that bearing capacity of the anchor could be further enhanced.

Recalling the past 50 years researching history of underreamed ground anchor, it can be found that most study objects were shallow underreamed anchors in early years. In recent years, the author and some scholars have found deep underreamed anchor has completely different failure modes and bearing mechanism from shallow anchor, and deep underreamed ground anchor should be the basic orientation in the engineering applications. To this end, it is necessary to give the classification criterion of these two types of underreamed anchor. Critical Embedment ratios (T/D)$_{cr}$ are always used for discriminant index, they are derived from different model tests, field tests and numerical analysis results. The use of critical embedment ratio (T/D)$_{cr}$ as classification criterion is reasonable. But in conditions of different soil, different dimensions of underreamed ground anchors and other factors, critical embedment ratios (T/D)$_{cr}$ may not be a certain value. According to present and previous research results, it is recommended that criterion between shallow and deep underreamed anchors should be the following range of values: (T/D)$_{cr}$=8.5-9.5.

4 Conclusion

A systematic research on the working behaviors of underreamed ground anchor, such as bearing characteristics and failure modes, was applied with model tests. And the conclusions are as follows.

1) According to the value of embedment ratio, the failure modes of underreamed ground anchor can be divided into two major types. The failure surface of the first mode is invert bell-shaped, it emerged to connect with the surface of the sand bed. And this type of anchor is called shallow
underreamed ground anchor. The second failure mode performed like a balloon, the rupture surface buried inside the sand bed. And this is deep underreamed anchor. The two types of anchors can be divided according to embedment ratio. The critical embedment ratio of underreamed ground anchor buried in medium sand under vertical load is 8.5.

2) The failure modes of the two types of underreamed ground anchor have completely different characteristics. After local shear failure, the bearing capacity of deep underreamed ground anchor will still increase slightly. While the bearing capacity of shallow anchor, after general shear failure, will decrease substantially.

3) By analyzing the failure modes of the two types of underreamed anchor, it is deemed that the failure mode of shallow anchor was general shear failure, so shallow underreamed anchor should be forbidden in practice. While deep anchor performed local shear failure, the failure surface would continue extending with load increasing. Therefore, the capacity of deep anchor would still increase after failure.

4) In practice, deep underreamed ground anchor has higher bering capacity, safety and economy. In engineering design, the condition of engineering geology, the dimensions and ultimate bearing capacity should be considered to adopt deep underreamed anchor in order to obtain greater economic benefits.

References


STABILITY ANALYSIS OF JOINTED ROCK TUNNEL USING GEOSMA-3D MODELING

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Analytical solution and simplified design procedure is presented for evaluating the response of existing joint rock tunnel. For joint rock, the stability of rock is controlled in a sense by the number of blocks, i.e. the size, orientation and locations of the discontinuities, especially the key blocks for a given excavation geometry. Key block failures occur where blocks of rock which are separated form the rest of the rock mass by discontinuities slide of fall into an excavation. In this research, according to block theory, the newly block stability analysis model is developed through GeoSMA-3D code. This model assumed that rock mass consists of blocks and each block is formed by particles arranged in a specific way, thus formulating a combination of block model and particle model. This code adopts vector analysis, which can simulate all excavation planes especially in the tunnel and other underground structure. It can also create three-dimensional structural model and analyze mobility of key block in the simulation plane by means of geometry and kinematics theory. The distribution of all key blocks and the quantitative data are analyzed by means of the newly developed program. It can be concluded that the newly code is a tool for modeling blocky rock masses.

1. Introduction

There are now many tunneling projects underway, including subway tunnels, railway tunnels, rapid railway tunnels, roadway tunnels, electricity tunnels, telecommunication tunnels, etc. The support design for tunnels is so far mostly empirical. The typical support design pattern is adopted based on the rock/soil types, or based on RMR (Rock Mass Rating) values. However, both stress- and structure-induced failures should be considered in the design of rock support for tunnel design for the assessment of the structure induced failures, the so-called block theory was suggested by Goodman and Shi (1985). Block theory is a useful tool to determine the removability and stability of rock blocks that were created by the intersection of joints. As pointed out in Jing and Hudson’s 2002 review paper on numerical methods in rock mechanics, the distinct element method (DEM) is one of the important numerical methods for simulating the mechanical properties of rock masses. Many researchers have paid attention to block deformation and failure in DEM. The updated DEM can be classified into two types: one is the particle model; and the other is the block model. This paper describes the development of a new numerical model, the block and particle model (BPM), to simulate both continuous and discontinuous deformations of rock masses. Both elastic deformation and failure of blocks can be described in this model. The model assumes that a rock mass consists of blocks and each block is formed by particles arranged in a specific way, thus formulating a combination of a block model and a particle model. Warburton (1981) suggested a computational procedure to analyze the stability of a single three-dimensional rock block. A block fracturing algorithm has been proposed by Lin (1995) in the context of rock facture, comprising again the Mohr-Coulomb fracturing criterion with a tension cut-off, where the newly formed discontinuities are introduced and are further treated in the same way as the original discontinuity planes.
In this paper, the probabilistic block theory, which was suggested by Hatzor and Goodman (1992), was applied to the example site. Their theory was summarized first, and their results were compared with ours. Moreover, failure shapes observed in the field were compared with the results obtained based on the block theory. Lastly, the individual key block stability analysis coded by the authors was performed using actual unrolled joint trace maps and compared with failure shapes observed on site.

2. Block theory in tunnel rock

In general, the analysis of tunnel stability using block theory is classified into two parts, the kinematic analysis and the stability analysis. In the kinematic analysis, discontinuities of rock masses are analyzed to determine whether the orientation of discontinuities could result in instability of the tunnel using the spherical projection technique. However, this analysis is restricted because it does not consider the loading conditions. Once it has been determined that a kinematically possible failure mode is present, a limit equilibrium stability analysis is performed to compare the resisting forces with the resultant forces. The stability analysis of a tunnel depends on several conditions, such as failure modes, loading conditions, block morphology, and analytical methodology.

According to the parameter, there will be two steps for the block searching process (Here the blocks are treated as tetrahedral blocks composed by three planes and the excavation face and the structural planes are infinite,. First, three groups of structural planes parameter will be selected at random and their normal vector and point of intersection will be calculated, if there is no point of intersection, the analysis process will be stopped and check out another combination. Second, we will calculate the intersection of the structural planes and test out whether it can intersect the excavation face of the tunnel if there do exist the intersection point. If not, the program will be stopped. In this way, we will find out all rock blocks around a tunnel and then can estimate the stability.

In order to model the deformation of a discontinuous zone, we adopt a non-linear mechanical model for discontinuities. In the case of shearing, the new model was used shearing rigidity of the discontinuous zone instead of penalty spring used. Therefore, shearing deformation from the zone can be simulated before shearing failure. Mohr-Coulomb’s law is used for shearing failure of two coupled blocks. If the shearing force calculated from the tangential spring gets larger than shearing strength \( \tau_m \) of the discontinuity, the tangential spring will be removed, and then, friction force takes the place of the spring. \( \tau_m \) is calculated by the following formula:

\[
\tau_m = C + f_n \cdot \tan \varphi
\]

(1)

Where \( C \) and \( \varphi \) are cohesion and friction angle of the discontinuity and \( f_n \) is the compressing force from the normal spring. According to this law, the safety factor( \( F \) ) is calculated below. Formula (2) is the situation of slipping along one side and (3) is along two sides.

\[
F = \frac{Q \cos \alpha \tan \varphi + CS}{Q \sin \alpha}
\]

(2)

\[
F = \frac{N_i \tan \varphi_i + N_j \tan \varphi_j + C_i S_i + C_j S_j}{Q \sin \alpha}
\]

(3)

Where \( Q \) is the weight of the key block, \( \alpha \) the dip of the slipping plane, \( S \) the area of the slipping
plane and $N_i, N_j$ are the normal force of the slipping planes.

\[
N_i = \frac{Q \cos \alpha \sin \alpha_i}{\sin(\alpha_i + \alpha_j)}
\]  

\[
N_j = \frac{Q \cos \alpha \sin \alpha_j}{\sin(\alpha_i + \alpha_j)}
\]

Where $\alpha$ is the dip of the intersection of the slipping plane $i$ and $j$. $\alpha_i$ and $\alpha_j$ are the angle between the upper normal of the plane and the vector of the weight vertical to the intersection.

2.2 Software developing tools

In this study, visual C++ is adopted to finish the mainframe of the program. C++ is the most popular object-oriented programming language and the object-oriented programming is the most popular technique. As for the graphic, there are many three-dimensional graphics development tools, the outstanding of which is openGL (open graphic library). We can simulate the reality of the tunnel perfectly with it, which means we can observe the key blocks in any angle (as shown in Figure 1). Also, we can move to any point of the tunnel dynamically by pressing the computer arrow keys. In addition, the code assumes the structural planes as disk shape which is finite.

![Diagram](image)

(a) Individual key blocks  
(b) Key blocks on the tunnel free face

Figure 1. Key blocks observation

3. Application of the Block Stability Analysis Model

Based on the above block model, a new software is developed by Northeastern University (China). One example is demonstrated to check the capability of the current model. The interested area is 100×50×100 m$^3$ with horizontal and vertical joints intersecting in the Dayaogou tunnel area.

Dayaogou Tunnel, the first large-span tunnel in northern Liaoning province, China, is scheduled to be completed in 2003. This tunnel is 460m in length, 21.242m in width and 15.52m in height (Wang, 2004). The rock formation of this tunnel is composed of thick-bedded muddy sandstone, thin alternations of sandstone and shale. The sandstone and shale alternation is less weathered and is not severely stained. However, the stability of this large-span tunnel is affected by faults or joints located nearby. Hence, a better understanding of the mechanics of influence, especially regarding the risk assessment of faults, is required. The influence of faults on the stability of underground openings has been investigated using our block stability analysis code in the following application.
3.1 Geology and joint survey and tunnel parameter

The joint survey is carried out in two sides in the entry of the tunnel and 93 structural planes are found. According to the field observation, the up-faulted rock is cut by joint sets very densely, that forms a crushed band. Based on the survey, the discontinuities are divided into 4 groups (Table 1).

<table>
<thead>
<tr>
<th>Set #</th>
<th>Dip(°)</th>
<th>Dip direction(°)</th>
<th>Max joint Trace (m)</th>
<th>Standard Deviation of dip(°)</th>
<th>Standard Deviation of Dip direction(°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>60</td>
<td>68</td>
<td>15.3</td>
<td>6.25</td>
<td>4.63</td>
</tr>
<tr>
<td>2</td>
<td>78</td>
<td>343</td>
<td>20.6</td>
<td>3.46</td>
<td>5.54</td>
</tr>
<tr>
<td>3</td>
<td>60</td>
<td>273</td>
<td>22.5</td>
<td>10.3</td>
<td>7.66</td>
</tr>
<tr>
<td>4</td>
<td>84</td>
<td>248</td>
<td>14.7</td>
<td>7.9</td>
<td>8.56</td>
</tr>
</tbody>
</table>

3.2 Results obtained from the block stability analysis software

Based upon the above analysis, here we will use the new code to search and identify the key blocks around the tunnel. The discontinuities used to analysis are listed in the table1. First, the fractures are simulated and added into the model. And then, key blocks are searched as is shown in Figure 2. According to the result obtained by the software, there are 286 combinations around the tunnel which must be carefully treated, including the situation of falling, slipping along one side, slipping along two sides. Table 2 shows some examples of the key blocks.

<table>
<thead>
<tr>
<th>Discontinuity number</th>
<th>Dip (°)</th>
<th>Dip direction(°)</th>
<th>Friction angle(°)</th>
<th>Friction Safe factor</th>
<th>Slipping type</th>
</tr>
</thead>
<tbody>
<tr>
<td>1#9</td>
<td>60</td>
<td>68</td>
<td>28</td>
<td>0.13</td>
<td>0.237</td>
</tr>
<tr>
<td>2#5</td>
<td>78</td>
<td>343</td>
<td>30</td>
<td>0.21</td>
<td>Slipping along 1#9 and 2#5</td>
</tr>
<tr>
<td>3#20</td>
<td>60</td>
<td>273</td>
<td>28</td>
<td>0.13</td>
<td>0.456</td>
</tr>
<tr>
<td>1#30</td>
<td>60</td>
<td>68</td>
<td>28</td>
<td>0.13</td>
<td>Slipping along 4#5</td>
</tr>
<tr>
<td>2#18</td>
<td>78</td>
<td>343</td>
<td>30</td>
<td>0.21</td>
<td></td>
</tr>
<tr>
<td>4#25</td>
<td>84</td>
<td>248</td>
<td>26</td>
<td>0.10</td>
<td></td>
</tr>
</tbody>
</table>

The number of the discontinuity is marked like this: 1#5 means the 5th plane of the 1 group. From these results, It can be found the difference of size of blocks also is rather large than measured results, it is more reality, because the joint planes are not extended infinitely and are cut each other. Obviously the failure of tunnel causes the nearby rock blocks sliding. If the tunnel can be strengthened before hand, may be the tunnel can avoid form sliding dropping.
4. Conclusions

Some aspects associated with the discontinuous deformation analysis framework in modeling rock tunnel have been discussed. In this paper, a new numerical simulation model is presented to simulate both continuous and discontinuous materials. The new code can simulate all discontinuities especially in the tunnel and other underground structures. Further, it can create three-dimensional structural modal and analyze mobility of block in the simulation plane by means of geometry and kinematics theory and the structural modal can be observed in any angle dynamically. Based on the field investigations, the block stability of a large scale tunnel in China is studied finally using the newly key block modeling technique.

It has been also demonstrated that the newly key block modeling technique is a tool to analysis blocky rock fracture deformation and failure follows a reasonable trend from above research, the magnitude of the fracture deformation computed is in general larger than that measured. Furthermore, in some cases the computed and measured fractures are of opposite sense. These findings point to the need to refine the simplifications and assumptions made in the analysis. To resolve this discrepancy between computed and measured results, additional refined analysis will be performed.

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References

ACOUSTIC EMISSION EXPERIMENT RESEARCH FOR ROCK MASS EXCAVATION

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Acoustic emission (AE) is a powerful nondestructive testing tool for examining the behavior of materials deforming under stress. It can be used as a stethoscope of fracture or damage for rock mass by listening to AE events when undergoing failure under the compressive loads. In this paper, an experimental study on the AE source location of the square cylinder granite specimens under uniaxial compression test was made. In order to obtain a three-dimensional location of AE events, eight AE sensors are mounted on the specimen. The AE source location was determined by acquisition of eight channel AE sensors after filtering, processing, reporting and visualizing seismic data. AE test provides an analysis of the microcracking activity inside the rock volume to predict jointed rock fracture patterns under uniaxial loading condition.

1. Introduction

With the development of national economic and defense construction, people gradually recognize that the uses of underground space play an important role in the improvement of ground environment, such as subway, tunnels and underground caverns. Take the collapse of Hangzhou Metro [1] in China, for example, the safety of the excavation disturbed or damaged zone (EDZ) around underground rock attracts more attentions. The study of damage formation in jointed or bulk rock under stress has been a subject of widespread interest, the results of which have led to a number of comprehensive texts [2-6]. The formation of the EDZ is affected by various factors, such as stress redistribution and excavation methods [7]. For many reasons, it is important to be able to predict the time, location and intensity of potential rock fracture. Fracture development in stressed rock has been observed extensively in the laboratory by a number of methods. Nowadays, as an effective approach, acoustic emission (AE) techniques are broadly applied to rock to obtain the information on crack initiation and propagation in rock engineering [8-11]. In this work, experimental investigations are carried out using the AE technique for characterizing the fracture behavior of square cylinder granite specimens of different geometry size. The acquisition of eight channel AE-data is analyzed to determine a three-dimensional location of AE-events.

In spite of the extensive work and the numerous successes in predicting rock fracture behavior, our comprehension of the physical processes that ultimately control fracture behavior is still weak [12]. Because of the heterogeneous inclusions (rigid or soft), the magnitude of local stress is significantly higher than the magnitude of applied stress, especially when the material is highly heterogeneous. As local stress and strength vary in a random fashion, the failure site in the material also varies randomly, and does not necessarily coincide with the maximum stress location unless a strong interactive stage between micro-fractures is reached [13]. Until now we do not have a good understanding of the effect of heterogeneity of rock materials on the subsequent progressive fracture process. It logically follows that if we have a better conception of heterogeneity, we will be in a better position to formulate predictive models and predict the rock fracture behavior. However, little attention has been paid to the detailed theoretical investigation of the influence of rock heterogeneity on progressive failure that leads
to collapse of rock mass under uniaxial compression.

2. **Acoustic Emission Experiment**

Acoustic emission characteristics of rock mass under deforming process reflect the whole failure or damage behavior, which is very complicated for theoretical and analytical explanation. In the laboratory experimental test, the complexity of AE events during rock failure or damage process is due to many factors, such as the composition and self-structure of rock material, the processing and precision of the specimens, loading control method and test environments. This is the main reason for the study on acoustic emission experiment that can not get the universal properties.

Despite this, there are some significant characteristics deriving from laboratory test under present conditions. Rock uniaxial compression test is always the indispensable part to measure the uniaxial compression strength (UCS) and study its damage mode and state. Hence, the study on rock acoustic emission is to observe its characteristics of AE data under uniaxial compression test. In the field of AE events under uniaxial compression test, extensive laboratory test researches are conducted among domestic and foreign scholars. In this section the AE experiment are implemented in Rock mechanics lab of Northeastern University, China.

2.1. **Samples preparation**

There are so many types of rock mass around the world. In order to make the test results preserve universal representativeness, this test picks the granite from Quyang of Hebei Province, China as experimental specimens, which is widely distributed in most of China provinces. The granite specimens for physical properties test are the square cylinder samples with 10cm in length and width respectively and 20cm in height. The sample preparation and testing procedure are according to the standard testing of International Society for Rock Mechanics (ISRM). The samples were prepared in the Experimental Centre of Rock Mechanics and Engineering at Northeastern University. The ratio of height to width or length of the specimens was 2, 2.14 and 1.5 respectively. Both ends of specimens were ground parallel to within 0.02mm and specimens were dried at 105° C before testing. Before install the probe, proper amount of butter should be smeared on the surface of the specimens, which is to ensure that the better AE data can be received by AE sensors.

2.2. **Testing equipments**

The loading system was performed using YAG-3000 Microcomputer Controlled Rock Stiffness Testing Machine, which is produced by Hangzhou Popwil Instrument CO., LTD. Its maximum loads for vertical and transversal direction are 3000kN and 400kN respectively, and the corresponding maximum compression testing displacement is 800mm and 220mm. The diameter of both pressure plates is $\phi$ 300mm with 1% testing precision.

At the same time, an Acoustic Emission System, Hyperion Seismic Software Suite, the production of ESG Canada Inc. (Engineering Seismology Group), was adopted to detect damage and fracture of rock samples. ESG’s Hyperion Seismic Software (HSS) package is a suite of programs designed for filtering, processing, reporting and visualizing seismic data. The modular Windows™ based software platform uses an open database design which allows users the ability to manage all aspects of seismic data flow, from acquisition and processing to detailed waveform analysis. Visualization and reporting are made easy with SeisVis™, an interactive 3D tool that outlays the characteristics of detected seismic
events (location, magnitude, source parameters) on a digital map of the specific mine, reservoir or structure being monitored.

And the data acquisition system, Ultrasonic Concrete Testing, purchased from Proceq UK Ltd., was used to record load and displacement. Ultrasonic Concrete Testing is based on the pulse velocity method to provide information on the uniformity of concrete, cavities, cracks and defects. The pulse velocity in a material depends on its density and its elastic properties which in turn are related to the quality and the compressive strength of the sample. It is therefore possible to obtain information about the properties of components by sonic investigations and applicable to AE test.

Fig. 1 shows the AE experimental system of rock uniaxial compressive test.

The diameter and the thickness of AE sensors (HagiSonic, Korea) were 3.6 and 2.4 mm, respectively. The main frequency band of the AE sensors was between 100 kHz and 1 MHz. To easily attach AE sensors to samples and obtain the same sensitivity for each sensor, an electron wax was used. AE signals measured in the sensors were amplified by 40 dB with pre-amplifiers (PAC model 1220A). Fig. 2 shows the sample before loading to which attached by sensors and strain gauge. The mechanical and acoustic emission (AE) measurements were conducted in the conventional uniaxial compressive test configuration. Fig. 1 shows the location of 8 sensors attached to the specimen during the uniaxial compressive test to perform the AE source location. Among them 4 sensors (numbered 1,2,3,4) are attached at the relative bottom of the sample in a horizontal plan, in the mean while, the other 4 sensors (numbered 5,6,7,8) are mounted at the upper part of specimen [28].
2.3. Experimental procedures

During the uniaxial compression test, the rate of displacement was controlled to 0.02 mm/min. In addition, the Ultrasonic Concrete Testing system manufactured was used for the AE measurements. Considering background noise, the AE trigger level was set to 40 dB. Time parameters for AE wave forms include Peak Definition Time (PDT), Hit Definition Time (HDT) and Hit Locking Time (HLT).

2.4. Experimental results

2.4.1. The AE location and rock failure pattern

Size effect of rock strength is not confined to the strength disparity of two different scale of rock mass. We can see from the failure mode clearly that different size of rock samples have the similar damage pattern, longitudinal splitting according to loading direction. Size effect does not have apparent influence on rock ultimate failure pattern, or in other words, basically have no effect at all.

2.4.2. AE characteristics in uniaxial compression

The AE location process of different size of rock specimens is shown in Fig. 3.

![AE location process](image)

50% of peak loading 75% of peak loading 100% of peak loading

Figure 3 AE location process G3

The AE events firstly appear in the smaller size of rock sample. And it is noteworthy that AE events are in a dispersed state at the initial stage of acoustic emission (50% of peak loading) rather than in the end part of sample compared with larger size of rock specimens. There are a large amount of AE events occur in the middle of the rock specimen, and with increasing of loading, which will be in the absolute dominate position relative to the end of specimen. At last, the increasing loading lead to the initiation and propagation of microcracks. Due to large quantity of microcracks, penetrating cracks appear, which result in the dispersed failure pattern.

As the relative larger rock specimens, at the early stage AE events occur only in the end of samples with the propagation of microcracks to the middle of them. Therefore, according to the experimental results, size effect indeed has a bit influence on AE location and the propagation mode of microcracks. There are explosion sound with fragments when all specimens fail.

In the ascending branch of the load-displacement curves, the AE characteristics of rock samples were similar:

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(1) During the initial loading stage, there was little occurrence of AE activities;
(2) As the load reached about 60% of the failure load, the AE activity became more intense;
(3) When the external load approaches the ultimate strength, the AE activity increased rapidly.

3. Conclusion

In this work, the measurements of acoustic emission and the source location were conducted on granite specimens under uniaxial compression. The practical particle flow problem seems simple, but actually complicated, and the mechanism is not well understood. It is necessary to use different models to analyze different materials. In this paper, introducing Burgers Model, the numerical analysis was carried out by using numerical tool PFC2D code coupled with rock characteristics of heterogeneity and anisotropy.

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