Prediction of mechanical parameters of the surrounding rock based on Hoek-Brown criterion

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Abstract: The research object is F1-6, a branch of Nujiang fault zone, which is crossed by the diversion tunnel of a hydropower station in Southwest of China. The relationship between geological strength index (GSI), disturbance parameter (D) and acoustic wave velocity of rock mass is explored. The Hoek-Brown criterion based on wave velocity is applied to predict the mechanical parameters of the surrounding rock in F1-6. The feasibility of the method is verified by the in-situ test of the fracture zone (F1-6), and the deformation of tunnel after excavated is predicted by the numerical simulation method. The study show that the mechanical parameters of fault fracture zone predicted by Hoek-Brown criterion based on wave velocity are close to the results of in-situ test. It is feasible to predict the mechanical parameters of deep fracture zone by this method, which can solve the difficulty that the mechanical parameters of deep fracture zone can not be obtained by test directly. In addition, the numerical simulation based on the derived mechanical parameters can provide guidance for the design and construction of tunnel engineering.

Keywords: the fault zone in Nujiang River; GSI; D; wave velocity; Hoek-Brown criterion; in-situ test; mechanical parameters; numerical simulation
1. Introduction

When a large underground cavern passes through the fault zone, how to obtain accurate and reliable mechanical parameters of the fault zone to provide guidance for the design and construction of underground engineering has always been the primary problem in the geotechnical engineering field, in addition to the research on activity of the fault zone. The main methods to obtain the mechanical parameters of the fault zone are as follows: the large-scale in-situ test, the laboratory test, engineering analogy, etc. The large-scale in-situ test is the most direct way, with the characteristics of accuracy and reliability. But it also has many disadvantages, such as long period, high cost and difficult operation; The laboratory test is limited to Size Effect, it is instructive for the determination of complete rock mechanics parameters, however, for the fault zone, there is a direct relationship with the sampling, and the measured results are often more discrete. The engineering analogy method has a high demand on the experience of geological engineers, which requires a full understanding of the engineering geological conditions and hydrogeological conditions of the fault zone to make an accurate judgment. Therefore, it is still difficult to determine the mechanical parameters of the fault zone, especially for the deep fault zone, it is more difficult to determine because of the difficulty of the large-scale in-situ test during investigation period. How to find a simple and feasible method to obtain the mechanical parameters of rock mass has become a hot topic in the academic circle, there are many experts and scholars have done related work. Hoek-brown Criterion is an empirical criterion for nonlinear failure of rock mass proposed by E. Hoek et al. in 1980 based on the reference of Griffith’s strength theory and through a large number of tests. It comprehensively considers the influence of rock mass structural characteristics on the strength, and is the most perfect method developed at present. The key to applying this criterion is to determine the GSI(geological strength index) and D(rock mass disturbance parameter). E. Hoek gives the generalized range of GSI and the generalized value of D \[1-3\], which are difficult to obtain and have deviation in engineering application. Su al.\[4\] quantified the GSI value by using RMR classification value, rock block index, weathering index, etc. Run et al.\[5\] determine the D value according to rock integrity index and rock damage variable. The above method can be used as the basis for determining GSI value and D value, but to get the above index in the fault zone can be difficult, so it is hard to generalize. Xia \[6\] found a simple way to determine the GSI parameters by studying the acoustic wave that characterizes the comprehensive characteristics of rock structural plane properties and rock integrity. The study shows that the rock strength parameters obtained by rock wave velocity method are similar to those obtained by Hoek’s recommendation method, and are applied in slope engineering. The conclusion of this method is lack of direct and effective verification by in-situ test and the error between this method and the actual project has not been further studied. Other scholars have done relevant research, and there are similar deficiencies\[13\]. When Song et al.\[7\] predicted the surrounding rock with a large minimum principal stress \(\sigma_3\) based on the Hoek-brown criterion of wave velocity of borehole, they found that the error was large. In addition, the study only focuses on the hard rock with higher quality, but not on the applicability of fractured rock mass. Based on the wave velocity method of Xia et al. And the in-situ test results of Nujiang Fault Zone Branch f1-6 in Tibet, this paper further verifies the correctness of the mechanical parameters of the fault zone inferred from Hoek Brown criterion of wave velocity in the hole, compares the error of the results, analyzes the causes and combined with the measured velocity in the hole. Furthermore, apply this method to predict the mechanical parameters of deep fracture zone.

2. Determination of mechanical parameter theory based on Hoek Brown criterion of wave velocity

2.1. Generalized Hoek Brown strength criterion

E. Hoek and E. T. Brown first proposed the narrow sense of Hoek Brown nonlinear empirical failure strength criterion \[8\] in 1980 through a large number of triaxial tests of rock blocks, and later improved in 2002 to put forward the generalized Hoek Brown nonlinear empirical failure strength criterion empirical formula \[9\], as shown in Equation 1.

\[
\sigma_1 = \sigma_3 + \sigma_c \left( m_b \frac{\sigma_1}{\sigma_3} + s \right)^a
\]

(1)

Where, \(\sigma_1\) and \(\sigma_3\) are the maximum and minimum principal stresses when the rock mass is damaged; \(\sigma_c\) is the uniaxial compressive strength of the rock mass; \(c, s\) and \(a\) are the material parameters of the rock mass, which are related to its lithology and structural plane conditions, and can be determined by the geological strength index GSI and the disturbance degree parameter D, as shown in Equation 2.
Where, GSI value is related to rock mass structure, which can be measured by outcrop in the field, estimated by RMR value, and calculated by rock mass wave velocity; D is the disturbance degree of rock mass caused by blasting failure and excavation unloading, and the value is 0 ~ 1. When rock mass is not disturbed or the disturbance is extremely small, D=0. According to the D value method proposed by Marinos et al. [1][10], the disturbance of rock mass around the drill can be ignored.

2.2. Relationship between wave velocity of rock mass and GSI and D values

Barton [11] obtained the relationship between the wave velocity $V_{mp}$ (km/s) of engineering rock mass and the quality index Q of rock mass through the statistics and summary of a large number of rock engineering data, as shown in Equation 3.

$$Q = 10V_{mp} - 3.5$$

In the early days, E. Hoek calculated the GSI value according to the geomechanical classification index RMR89, as shown in Equation 4, but the index RMR89 value needs to rely on the statistical situation of joints and fissures, which is obviously not applicable in drilling holes. Barton [9] also proposed the relationship between RMR89 and Q, as shown in Equation 5.

$$GSI = RMR_{89} - 5(RMR_{89} > 23)$$

$$RMR_{89} = 15\log Q + 50$$

Substituting Eq. 3 and Eq. 5 into Eq. 4, the relationship between rock mass wave velocity $V_{mp}$ (km/s) and GSI parameters is obtained [6], as shown in Eq. 6.

$$GSI = 15V_{mp} - 7.5$$

The $V_{mp}$ index in the formula is the undisturbed rock wave velocity, and its value should be greater than 1.700km/s. When the rock mass is disturbed by the near blasting, the GSI index obtained in Formula 6 has a large error. Therefore, in this study, the disturbance to the sample should be minimized when measuring the wave velocity. Lu et al. [12] established the relationship between BQ and GSI parameters according to Equation 6, as shown in Equation 7.

$$GSI = 15V_{cp}\sqrt{(BQ - 3\sigma_{cw} - 90)/250} - 7.5$$

Where, $V_{cp}$ is the P-wave velocity of the rock block and $\sigma_{cw}$ is the saturated uniaxial compression strength of the rock block. This formula provides another way to get GSI parameters.

As for the value of rock disturbance parameter D, Wu and Xu [13] considered the relationship between the effective elastic modulus before and after rock disturbance, and introduced rock wave velocity to determine rock disturbance parameter D. Xia et al. [6] summed up the shortcomings of the predecessors and deduced the estimation formula of D value again, as shown in Equation 8.

$$D = 2\left[1 - \frac{\frac{10(V_{mp} - 3.5)^{3/5}}{10(V_{mp} - 3.5)^{3/5}}}{10(V_{mp} - 3.5)^{3/5}}\right]$$

In the formula, $V_{mp}$ is the undisturbed P-wave velocity of the rock mass, $V_{mp}$ is the disturbed P-wave velocity of the rock mass; Then, lead into the Rs parameter (the wave velocity decreasing degree of rock mass disturbed), and the Expression 9 is obtained.

$$R_s = 10(V_{mp} - V_s)^3$$

If Formula 7 and Formula 8 are combined, the value of D can be expressed as Formula 10.

$$D = 2\left[1 - R_s\right]$$

Where: when the rock mass is not disturbed or disturbed extremely, $R_s = 1$, $D= 0$; when the rock mass is disturbed strongly, $R_s = 0.5$, $D = 1.0$; when $R_s < 0.5$, $R_s = 0.5$. 

$$m_b = m_i \left\{ \frac{GSI - 100}{28 - 14D} \right\}$$

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

$$a = \frac{1}{2} + \frac{1}{6}(e^{-GSI/15} - e^{-20/3})$$

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Substituting Eq. 6 and Eq. 10 into Eq. 2, we can get the relation between, \(m_b\), \(s\), \(a\) and rock wave velocity, see Eq. 11 [6].

\[
m_b = m \exp\left(\frac{15V_{ad} - 107.5}{28R_s}\right)
\]

\[
s = \exp\left(\frac{15V_{ad} - 107.5}{3 + 6R_s}\right)
\]

\[
a = \frac{1}{2} + \frac{1}{6} \left(\frac{7.5 - 15V_{ad}}{15} - e^{-20/3}\right)
\]

2.3 The method of determining mechanical parameters by Hoek Brown criterion

For tunnel engineering, the mechanical parameters of surrounding rock are mainly concerned with shear strength, elastic modulus, Poisson's ratio and deformation modulus, etc. the mechanical parameters of this fault zone study are shear strength and deformation modulus.

According to Hoek-Brown criterion, the expressions of internal friction angle \(\varphi\) and cohesion \(c\) of equivalent shear strength index of rock mass are respectively Formula 12 and Formula 13.

\[
\varphi = \sin^{-1}\left[\frac{6am_b(s + m_b\sigma_{3n})^{a^{-1}}}{2(1+a)(1+2a) + 6am_b(s + m_b\sigma_{3n})^{a^{-1}}}ight]
\]

\[
c = \frac{\sigma_c\left[(1+2a)s + (1-a)m_b\sigma_{3n}\right]^{a^{-1}}}{(1+a)(2+a)\sqrt{1 + \left[6am_b(s + m_b\sigma_{3n})^{a^{-1}}\right]/[(1+a)(2+a)]}}
\]

Where, \(\sigma_{3n} = \sigma_{3\text{max}} / \sigma_c\), parameters \(m_b\), \(a\) and \(s\) are calculated from Formula 2. Maximum minimum principal stress \(\sigma_{3\text{max}}\) is determined by in-situ stress measurement, and \(\sigma_c\) is the compressive strength of rock blocks. For the fault zone, it is reduced according to the compressive strength of the original rock.

The expression of elastic modulus \(E_m\) of rock mass determined by wave velocity is Eq. 14.

\[
E_m = \begin{cases} 
R_1 \sqrt{\frac{\sigma_c}{100}} & (\sigma_c \leq 100\text{MPa}) \\
R_1^{15V_{ad} = 17.5} & (\sigma_c > 100\text{MPa}) 
\end{cases}
\]

3. Engineering application

3.1 Project overview

This time, a power station project in Yuqu River, Tibet, is taken as the research object. The dam and powerhouse are located on both sides of the river bend, and are connected by a diversion tunnel under pressure with a length of several kilometers. This diversion tunnel can not avoid crossing regional faults F1-6. Due to the wide fault zone, fractured rock mass and weak rock mass, stability of surrounding rocks may exist. Therefore, the accurate determination of the physical and mechanical properties of the fault zone has important guiding significance for line layout and structural design of diversion tunnel and treatment of geological defects of surrounding rocks. In this study, the wave velocity was used to predict the physical and mechanical parameters of the fault zone, which was verified by in-situ experiments.

The regional fault zone F1-6 extends along the Naozhong Gully as shown in Figure 1. It is exposed in the gully and at the toe of the gully. The western plate (lower plate) is Permian slate, marble, limestone or metamorphic rhyolite porphyry, while the eastern plate (upper plate) is Permian slate. The fault surface of the inner side of the NaoZhong Gully winding mountain highway has a tendency of 116° and a dip Angle of 47°. The visible width of the fault zone is only about 10m, and the rest is covered by the material at the bottom of the ditch. It is inferred that the width of fracture zone is about 230m and the width of influencing zone is more than 250m. Hot springs are exposed at the toe of the gully, which is a regional active fault zone. The diversion tunnel passes through fracture F1-6, the length of the tunnel in the fracture zone is 367 m, and the depth of the bottom plate...
is 117-190 m, as shown in Figure 2.

![Spatial distribution diagram of regional fracture F1-6](image)

**Fig. 1 Spatial distribution diagram of regional fracture F1-6**

![Cross section of fracture F1-6 at diversion tunnel](image)

**Fig. 2 Cross section of fracture F1-6 at diversion tunnel**

1. Quaternary colluvium; 2. The fourth member of Manghua formation in mid—Triassic; 3. The fourth layer of the first member of Nacuo group in lower Permian system; 4. Gravelly soil; 5. Calcareous slate; 6. Metamorphic rhyolite porphyry; 7. Boundary between the quaternary and bedrock; 8. Lithologic boundary (Dotted line is speculation); 9. Fault and its number (Dotted line is speculation); 10. Fracture zone of cataclastic flake rock; 11. Fracture zone of cataclasite; 12. Influence belt of faults; 13. Drilling and its number; 14. Trial pits and its number; 15. Attitude of the stratum of the rock or faults; 16. Designed diversion tunnel

3.2 Determination of physical and mechanical properties of fault zone

The regional fault F1-6 fracture zone consists of cataclastic Schistose rock and cataclasite (original rock is the marble). The cataclastic Schistose rock is dark grey and grey black, and the original rock is mainly sandy slate (see Figure 3). Dry drilling is difficult. The core of mud drilling is debris, breccia with rock debris and rock powder (see Figure 4). The recovery of drilling is 23%-90% and achievement rates is 0. In the process of drilling, the hole wall often breaks. In this study, the cataclastic Schistose rock is selected as the research object, and the results of in-situ test are compared with those of predicted strength by wave velocity test.
The depth of in-situ test pit for schistose broken rock is about 5.0m, and the thickness of surface colluvium gravelly soil is 0.5-1.0m, which is mainly composed of broken schistose rock. The depth of test point is 4.0-5.0m (see figure 5 and figure 6). A total of 8 shear test points (typical specimen is shown in figure 7) and 3 deformation test points (typical specimen is shown in figure 8) were performed on site. The shear test $\tau - \sigma$ curve is shown in Figure 9, the deformation test curve is shown in Figure 10, and the test results are shown in Table 1 and table 2. $Q_{\text{col+dl}}$
Fig. 9 Curves of τ-σ by shear test in TK3 test pit

Fig. 10 Deformation test curves of TK3 test pit

Table 1 In-situ shear test results in F1-6 fractured schist

| Number of test pit | Number of pilot | Normal stress (MPa) | shear strength (MPa) | shearing stress (MPa) | friction coefficient f | cohesion C' (MPa) | frictional strength |
|-------------------|----------------|---------------------|---------------------|----------------------|-----------------------|------------------|-------------------|
| TK3               |                |                     |                     |                      |                       |                  |                   |
| τ2-1              | 1.33           | 1.13                | 1.02                |                      |                       |                  |                   |
| τ2-3              | 1.00           | 1.17                | 1.11                |                      |                       |                  |                   |
| τ2-5              | 0.39           | 0.63                | 0.66                |                      |                       |                  |                   |
| τ2-6              | 1.17           | 1.13                | 1.12                |                      |                       | 0.61             | 0.39              |
| τ2-8              | 0.89           | 0.77                | 0.76                |                      |                       |                  |                   |
| τ2-2              | 1.67           | 0.95                | 0.87                |                      |                       | 0.50             | 0.11              |
| τ2-4              | 0.67           | 0.42                | 0.44                |                      |                       |                  |                   |
| τ2-7              | 0.56           | 0.42                | 0.41                |                      |                       |                  |                   |

Table 2 In-situ deformation test results in F1-6 fractured schist
3.3. Prediction of mechanical parameters based on wave velocity

The results of acoustic wave testing in the hole of the f1-6 hole of the fault zone (the interval between test points is 0.5m) were statistically analyzed, and the acoustic wave velocity of the ground surface buried depth section of the broken flake rock was obtained at 3 ~ 8m (in situ test depth section) and the body section of the tunnel (elevation 2775 ~ 2785m). At the same time, the undisturbed weakly weathered and slightly weathered sandy slate of the same formation lithology near the fault zone was tested with sound wave velocity. The results are shown in Table 3.

| Number of test pit | Normal stress (MPa) | deformation modulus(GPa) | Mean value of deformation modulus(GPa) | elastic modulus(GPa) | Mean value of elastic modulus(GPa) |
|-------------------|---------------------|--------------------------|----------------------------------------|----------------------|-----------------------------------|
| TK3               | E2-1                | 1.0                      | 0.178                                  | 0.579                | 0.552                             |
|                   | E2-2                |                          | 0.169                                  |                      |                                   |
|                   | E2-3                |                          | 0.109                                  |                      |                                   |

Table 3 Wave velocity test results of fractured schist in fault zone F1-6.

| Measuring position | Measured value of acoustic wave velocity(m/s) | Number of statistical samples |
|--------------------|-----------------------------------------------|------------------------------|
| Surface section    | Minimum 2041, Maximum 2667, Average 2299     | 22                           |
| Tunnel section     | Minimum 2410, Maximum 3700, Average 2870     | 42                           |

Since the velocity of wave is determined in borehole, the disturbance degree of borehole to rock mass can be neglected, then the descent degree parameter Rs is taken as 1, and the disturbance parameter D of rock mass is taken as 0. According to the results of in-situ stress test in the area of water diversion line, the horizontal stress plays a leading role in the in-situ stress field. The maximum horizontal principal stress $\sigma_H = (1.3 \sim 1.4)\sigma_z$, $\sigma_H > \sigma_h = \sigma_z$ ($\sigma_h$ is the minimum horizontal principal stress, $\sigma_z$ is the vertical stress), the orientation of maximum horizontal principal stress determined is $59^\circ \sim 80^\circ$. The lithological coefficient $m_i$ is taken as 9 according to the rock type as SLATE. Then the Hoek-Brown criterion based on wave velocity predicts the mechanical parameters of the broken zone, as shown in Table 4.

Table 4 Prediction of mechanical parameters in fractured zones based on Hoek-Brown criterion and wave velocity.

| Measuring position | $v$(m/s) | GSI | $\sigma_c$(MPa) | $\sigma_{min}$(MPa) | $m_b$ | $s/10^{-11}$ | $a$ | $\varphi$ | $E_{em}$(GPa) |
|--------------------|----------|-----|----------------|--------------------|-------|---------------|----|----------|--------------|
| Surface section    | 2299     | 22.5| 6              | 1.00               | 0.63  | 2.5           | 0.53 | 0.15     | 0.48         | 0.652        |
| Tunnel section     | 2870     | 34.5| 6              | 2.88               | 0.87  | 6.9           | 0.52 | 0.34     | 0.37         | 1.004        |

Note: (1) $v$: Wave velocity, the wave velocity adopted in this calculation is the average wave velocity $v_{ave}$; (2) $\sigma_c$: Uniaxial compressive strength, the uniaxial compressive strength of the rock block is the measured value of uniaxial compressive strength of the block taken out of the fault zone. (3) $\sigma_{max}$: the maximum value of the minimum principal stress, according to the measured Poisson ratio of 0.5, the side pressure coefficient $k=1.0$ is calculated for the surface section. The positive stress of in-situ test is 1MPa, and that of $\sigma_{max}$ is 1MPa. The vertical stress of the tunnel body is taken as lead and the buried depth is 120m. The test results show that the gravity is 23 KN/m², then $\sigma_{max}$ is 2.88 MPa.

3.4. Constrictive and analysis of the results

Comparison between in-situ test of rock mass in surface section fracture zone and prediction results based on Hoek-Brown criterion of velocity is shown in Table 5.

Table 5 Comparison of in-situ test results with prediction results based on wave velocity and Hoek-Brown criterion.

| Determination method | Shearing stress | Elastic modulus |
|---------------------|-----------------|-----------------|
| In-situ test        | $0.10 \sim 0.35$| $0.48 \sim 0.60$| $0.552$        |
The results are as follows:(1) The mechanical parameters of the rock surface segment of the fault zone obtained by the two approaches are close to each other. The shear strength parameters predicted by Hoek Brown criterion based on acoustic wave are close to the lower limit value of in-situ test, which may be caused by the comprehensive influence of wave velocity on the reaction rock mass, including the groundwater factor, while the in-situ test does not consider the factors of groundwater, resulting in a large result. (2) The prediction and calculation results of Hoek-brown criterion based on wave velocity show that the cohesion c increases with the increase of depth, and the change is obvious, while the friction coefficient f tends to decrease as the depth increases, this is because for weak rock mass, with the increase of surrounding rock stress, it will show a trend of transformation from elastic body to plastic body. Therefore, it is theoretically possible that the cohesion of the rock mass in which the tunnel is located will increase and the friction coefficient will decrease by using Hoek Brown criterion; (3) According to the research of Song [7], for the surrounding rock with high minimum principal stress, the prediction results of Hoek Brown criterion based on wave velocity have a large error with the empirical value in the specification. However, for the weak rock mass with small strength and confining pressure in this study, the prediction results of Hoek Brown criterion are close to the empirical values, and this study further verifies the research results of Song Yanhui.

4. Conclusion
By comparing the in-situ test results of rock mass in the fault zone with the results of rock mass mechanics calculated by Hoek Brown criterion based on wave velocity, the following conclusions are obtained:
(1) For bedrock with various structural planes, laboratory tests are limited by the size of the sample. Although the in-situ test is more accurate and reliable, it is not easy to be widely carried out because of its high cost and long period, and it is more difficult to carry out in-situ test for deep buried rock mass. The borehole acoustic wave is the characterization of the comprehensive characteristics of rock mass. The method of calculating the mechanical parameters of rock mass based on Hoek Brown criterion of wave velocity can make up for the deficiency of indoor rock test and in-situ test during the exploration period, which has certain guiding significance for predicting the mechanical parameters of rock mass.
(2) For the slates with developed fracture poles, the prediction based on Hoek-brown criterion of wave velocity is generally close to the mechanical parameters of rock mass obtained from in-situ tests in terms of strength and deformation. The strength parameter calculated by Hoek Brown criterion based on wave velocity is the lower limit of in-situ test results, which is rather safe. The results show that the Hoek Brown criterion calculation method based on wave velocity can be used as the basis for the parameters of rock mass in fault zone.
(3) Using Hoek-brown criterion based on wave velocity to estimate the mechanical strength parameters of underground surrounding rock has the advantages of being fast, efficient and low cost, which has certain guiding basis for underground engineering investigation, design and construction. It is suggested that the mechanical parameters of surrounding rock should be comprehensively evaluated in combination with the engineering geological analysis method and specifications.
(4) The strength parameters of underground surrounding rock are not only controlled by the strength characteristics of rock mass and structural plane, but also related to external factors such as surrounding rock stress and groundwater. The relationship between these external factors and rock acoustic wave velocity needs further study, only by making clear of these relations can we better use Hoek Brown criterion based on wave velocity to calculate the mechanical parameters of rock mass.

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