Research Article
In Situ Needle Penetration Test and Its Application in a Sericite Schist Railway Tunnel, Southwest of China

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The sericite schist is a typical metamorphic soft rock. Large deformation of surrounding rock often occurs in the construction of a tunnel in this stratum. Due to the broken rock mass structure and poor mechanical strength in Baishitou tunnel project of Dalin line of Southwest railway, it is impossible to prepare standard samples for a traditional rock mechanical test. Therefore, we chose penetrometer (SH-70) for an in situ test. Firstly, we monitored the deformation of typical sections and analyzed the characteristics of large deformation of soft rock in the tunnel. Secondly, we tested the needle penetration index of fresh excavation face and side wall. Then, we estimated some mechanical parameters of sericite schist by a needle penetrometer and Hoek-brown criterion and discussed the acquisition of mechanical parameters of soft rock. The results show the following: (1) The characteristics of extrusion rock tunnel are summarized as large deformation, fast deformation rate, and obvious construction disturbance. (2) The reference value of penetration index of sericite schist (the vertical joint direction) is 3.90~7.77 N/mm, and the parallel joint direction is 1.27~2.99 N/mm. (3) The uniaxial compressive strength estimated by a penetrometer is 0.78~8.53 MPa, and the strength of the surrounding rock is negatively correlated with the amount of deformation. Therefore, it can be considered that the insufficient strength of surrounding rock is the fundamental reason for large deformation. (4) The reference value of cohesion of sericite schist estimated by a penetrometer is 0.203 MPa, and the reference value of internal friction angle is 18.22°. Compared with the common estimation methods, the penetrometer is more convenient and economical, which can provide a new idea for obtaining the mechanical parameters of sericite schist soft rock tunnel.

1. Introduction

Large deformation of weak surrounding rock is a common phenomenon in underground engineering construction. In the construction of a railway tunnel in Southwest China, large deformation often leads to steel arch deformation, concrete cracking, and lining failure.

Sericite schist is a typical metamorphic soft rock. The research on the engineering mechanical properties of the rock is relatively mature in the industry, but the indoor test and research on its mechanical parameters are often time-consuming and laborious. It is convenient and fast to obtain the mechanical parameters of soft rock and put forward reasonable and effective large deformation control measures as soon as possible, which can reduce the risk of tunnel construction to a certain extent.

In the field acquisition of rock mechanical parameters, point load test and Hoek-Brown criterion are commonly used methods. For example, Zhang et al. [1] used the comparison results of point load instrument and indoor rock compressive strength testing research in the rock engineering geological exploration of open-pit coal mine and gave...
the relationship between point load strength of soft rock and compressive strength of rock through radial and axial point load strength testing of rock. Wang [2] analyzed the relative error between the uniaxial compressive strength converted by the point load test and indoor standard rock samples and then discussed the practical value of the point load test of irregular rock blocks. Wang [3] believed that the point load strength test was an efficient and effective means of rock testing. Hu et al. [4] introduced joint volume number of rock mass, joint condition, and quantitative geological strength index to establish the surrounding rock classification system. Hoek-Brown strength criterion was used to estimate the mechanical parameters of rock mass and combined it with specific engineering to estimate the mechanical parameters of the rock mass to verify the application effect. Li et al. [5] calculated the main mechanical parameters of engineering rock mass through rock mechanics test data and field geological survey results, according to Hoek-Brown failure criterion and equal area principle. The calculated result is also applicable in rock mass classification of underground engineering. Xia et al. [6] established estimation formulas of geological strength index and rock mass disturbance parameter, according to rock mass wave velocity. Then, they used the Hoek-Brown criterion to estimate rock mass mechanical parameters of the Lancang River transdomain engineering slope of the China-Myanmar oil and gas pipeline. Jiang et al. [7] took a lead-zinc ore body as an example and proposed an improved geological strength index value quantification and correction method. According to the above method and Hoek-Brown strength criterion, they determined the physical parameters of ore and rock. Then, compared with the field deformation test, the accuracy and feasibility of this method are verified.

The above method has some limitations for soft rock with very low strength. Therefore, a needle penetrometer is a good choice. Ngan et al. [8, 9] used the needle penetrometer test successfully to qualitatively distinguish carbonate sands from very weak and weak calcarenites in borehole cores recovered for cut-and-cover tunnel projects in Maastricht. Aydan [10] correlations between the NPI are explored, and several empirical relations are presented to infer various geomechanical properties, such as water content, elastic modulus, uniaxial compressive strength, Brazilian tensile strength, elastic wave velocity, friction angle, and cohesion. Li et al. [11] calculated the uniaxial compressive strength of different soft rocks by in situ and laboratory tests with a needle penetrometer; then, they compared the results of the needle penetration test and point load test of rock samples. The results show little difference between the two methods, indicating that it is reasonable and feasible to test the strength of soft rocks with a needle penetrometer, which has a good popularization value in soft rock engineering. Kahraman [12] used point load and needle penetration index tests and carried out the conversion factor for the UCS-NPI ratio of coal. Dipova [13] established a new database based on new laboratory test results, and an empirical relationship was developed to estimate the UCS from NPR stabilized soils and soft rocks. Attempts were made to contribute literature on the application of the NP test in some geo-materials, including compacted soils, chemically stabilized soils, soft tufa facies, and pumice. Rahimi et al. [14] estimate the UCS of gypsum rock specimens based on samples obtained from various locations in Iran, using the NP test. The conversion factors for dry and saturated gypsum rocks were 0.49 and 0.56.

The fundamental reason for the large deformation of tunnel is the weakness of surrounding rock. But sericite schist in the study area is impossible to make a sample to test in laboratory because of its thin layer, rapid weathering, and rapid decline of water strength. In this paper, we analyze the characteristics of large deformation of Baishitou tunnel, take the in situ test, obtain the NPI index of sericite schist, estimate the basic mechanical parameters of sericite schist, and discuss some methods of soft rock strength test.

2. Geological Environmental Conditions

2.1. Project Overview. The location of Baishitou tunnel is in Dalin Railway (Dali-Lincang) between Yunxian station and Toudaoshui waiting station, with a length of 9375 m. The Baishitou tunnel has undulating landforms, and the buried depth is between 8 m and 310 m. The tunnel is constructed according to the principle of New Austrian Tunneling Method: advance small conduit grouting, excavation of reserved core soil at three steps, combined support of shotcrete anchor steel arch frame, and one-time pouring of inverted arch secondary lining [15, 16]. During the construction of the Baishitou tunnel, the most intuitive manifestation of large deformation is the invasion of surrounding rock, large deformation, and failure of the steel arch frame. The deformation photos of typical tunnel sections are shown in Figure 1.

2.2. Formation Lithology. According to the geological report, the Baishitou tunnel is mainly distributed in the lower Paleozoic Lancang Group (P2ln) sericite schist, sericite-quartz schist carbonaceous sericite schist, and chlorite schist. Most of them are thin-bedded, and under the influence of structure, folds commonly developed. According to the photos exposed by site excavation (Figure 2), there are weak interlayers such as thin carbonaceous sericite schist and chlorite schist in the strata, which soften quickly after being exposed to water and have insufficient strength.

Affected by the tectonic movements of Honghe fault zone, Weishan fault zone, Lancang River fault zone, Pu’er fault zone, and Nanling River fault zone, the rock mass joints and fissures in the tunnel site are developed, which reveals that the rock core is developed with high angle joints and fissures, and the rock mass is relatively broken.

According to the formation lithology, address structure, and the surrounding rock exposed on site, the fundamental reason for the large deformation of tunnel is the weakness of the surrounding rock.

3. Deformation Monitoring and Analysis of Typical Tunnel Sections

The vault subsidence and horizontal displacement are the most intuitive responses to tunnel deformation. We adopt...
Figure 1: Large deformation of the tunnel. (a) The steel arch was twisted and broken. (b) Tilt limit of surrounding rock on the left.

Figure 2: Field surrounding rock conditions. (a) Sericite schist (Pz1,1n). (b) Sericite schist interblended with quartz.

Figure 3: Layout of deformation monitoring points.
the method shown in Figure 3 to arrange monitoring points to monitor the settlement of the crown and the horizontal displacement around. The deformation time history diagram and deformation rate diagram of typical section DK163-70 are shown in Figure 4.

From the 15-day deformation time history and deformation rate diagram of DK163-70, the cumulative deformation of the vault is 355 mm, and the maximum deformation rate is 42 mm/d. The cumulative deformation of the left arch is 670 mm, and the maximum deformation rate is 78 mm/d. The cumulative deformation of the right arch is 337 mm, and the maximum deformation rate per day is 35 mm/d. The cumulative deformation of the left sidewall is 359 mm, and the maximum deformation rate is 118 mm/d. The cumulative deformation of the right sidewall is 140 mm, and the maximum deformation rate is 37 mm/d.

The vault settlement and arch foot horizontal displacement gradually tend to be stable when the initial support is closing, the deformation rate is large when the upper bench excavation to the lower bench excavation then gradually tends to zero when the initial support is closing. Therefore, the characteristic extrusion of Baishitou tunnel can be summarized as follows: large deformation, large deformation rate, and obvious construction disturbance.

4. In Situ Test of Needle Penetration

4.1. Introduction of a Needle Penetrometer. The instrument used in this experiment is the SH-70 needle penetrometer manufactured by Maruto Company in Japan (Figure 5). A needle penetrometer is a device for obtaining the needle penetration index (N/mm) in the field. The advantages of the penetrometer are as follows: it is easy to carry and use; there is no need to prepare standard samples, and it can be used for nondestructive testing of in situ and indoor rocks; and many researchers show that the instrument used in soft rock strength testing has a good effect. However, the disadvantages of the penetrometer are as follows: the penetrating needle can only be inserted into the rock surface to a depth of about 10 mm, which cannot fully represent the physical properties of the rock as a whole; and due to the differences between types or regions, the results estimated by this device may be different.

It can quickly convert the uniaxial compressive strength. The needle penetration index and the uniaxial compressive strength calculation formulas are as follows:

\[ D = 10 \text{ mm}, F \leq 100 \text{ N} : \text{NPI} = \frac{F}{10}, \]

\[ F = 100 \text{ N}, D \leq 10 \text{ mm} : \text{NPI} = , \]

\[ \log \text{UCS} = 0.978 \log \text{NPI} + 2.621, \]

where \( D \) is the penetration volume (mm) and \( F \) is the penetration force (N).

4.2. In Situ Test of the Fresh Surrounding Rock. A total of 1057 groups of in situ tests of surrounding rock at small mileage DK163 + 066 ~ 076, large mileage DK164 + 84.6 ~ 106, and DK164 + 138 ~ 147 are completed using the needle penetrometer. The test sites are in the freshly excavated tunnel face, side walls, with 726 groups of vertical joint faces and 331 groups of horizontal joint faces. The field test photos are shown in Figure 6, and the results of needle penetration test are shown in Table 1.

As seen from Table 1, the average needle penetration index in the vertical joint direction of surrounding rock is 3.90~7.77 N/mm. The average needle penetration index of the parallel joint direction is 1.27~2.99 N/mm. During the test, it is found that the needle in the parallel direction can easily penetrate into the rock, resulting in the needle
penetration index in parallel direction which is much smaller than that in the vertical direction. Due to the anisotropy of the rock, the needle penetration index in the vertical direction is about 1.81~3.6 times than that in the parallel direction.

The uniaxial compressive strength of the tunnel face and sidewall from each test section obtained by the needle penetrometer are shown in Table 2. In DK163 + 076 ~ 066, UCS⊥a is 1.89 MPa, and UCS∥a is 0.53 MPa; the UCS⊥a on the left and right sides are close, slightly larger than the UCS⊥a on the tunnel face; in DK164 + 084.6 ~ 106, UCS⊥a is 1.58 MPa, and UCS∥a is 0.88 MPa. UCS⊥a of the left wall is larger than that of the right wall, while the opposite is true in the parallel direction. UCS⊥a of the left side of the tunnel face is slightly larger than that of the right and middle sides; in DK164+138~147, UCS⊥a is 3.13 MPa, UCS∥a is 1.22 MPa, and UCS⊥a in the middle of the tunnel face is much larger than UCS⊥a on the left and right sides.

The deformation of the left wall is significantly greater than that of the right wall, based on the Table 2, the uniaxial compressive strength of the surrounding rock of the left wall of section DK163 76~66 is 2.05 MPa, and the uniaxial compressive strength of the right wall is 2.55 MPa. It is proved that the parts with low relative strength of surrounding rock are more likely to have large deformation, and the unbalanced stress of steel arch frame will lead to serious deformation and failure.

The uniaxial compressive strength was obtained by the needle penetrometer with other literature as shown in Figure 7. The empirical functions are shown in Table 3. As can be seen from Figure 5, most of the other data are located near the 1:1 straight line. When it is less than 5 MPa, the dispersion is small, and the empirical models show high similarity. It shows that the needle penetrometer is feasible in predicting the strength of general soft rocks. Especially for sericite schist, which is difficult to sample and has very

### Table 1: Results of the needle penetration test.

| Test sections | Vertical joint direction (group) Value (N/mm) | Parallel joint direction (group) Value (N/mm) |
|---------------|---------------------------------------------|---------------------------------------------|
| DK163 + 066 ~ 076 | 256 | 4.67 | 51 | 1.27 |
| DK164 + 84.6 ~ 106 | 253 | 3.90 | 63 | 2.15 |
| DK164 + 138 ~ 147 | 217 | 7.77 | 217 | 2.99 |

Figure 5: Needle penetrometer.

Figure 6: Photos of in situ field testing. (a) In situ test of tunnel face. (b) In situ test of sidewall.

(a) (b)
TABLE 2: Statistical table of UCS of tunnel face and sidewall.

| Section position         | DK163 + 076 ~ 066 | DK164 + 084.6 ~ 106 | DK164 + 138 ~ 147 |
|--------------------------|-------------------|---------------------|-------------------|
|                          | Groups            | UCS\(_\perp\) a (MPa) | UCS\(_\perp\) a (MPa) | Groups            | UCS\(_\perp\) a (MPa) | UCS\(_\perp\) a (MPa) | Groups            | UCS\(_\perp\) a (MPa) | UCS\(_\perp\) a (MPa) |
| Left wall                | 86                | 2.05                | /                  | 43                | 2.29                | 0.75                | 100               | 1.61                | 1.24                |
| Left                     | 42                | 1.43                | 0.95               | 56                | 2.03                | 0.93                | 86                | 1.58                | 1.20                |
| Tunnel face              | Middle            | 42                  | 1.48                | 0.47              | 67                  | 1.17                | 0.30              | 72                  | 7.24                | 1.29                |
| Right                    | 95                | 1.93                | 1.81               | 66                | 1.09                | 0.70                | 80                | 3.85                | 1.15                |
| Right wall               | 42                | 2.55                | 1.72               | 84                | 1.31                | 1.72                | 96                | 1.63                | 1.24                |
| Vertical joint direction | 256               | 1.89                | /                  | 253               | 1.58                | /                  | 217               | 3.13                | /                   |
| Parallel joint direction | 51                | /                   | 0.53               | 63                | /                   | 0.88               | 217               | /                   | 1.22                |

\(\ast\) The UCS\(_\perp\) a is the average value of uniaxial compressive strength in the vertical joint direction. The UCS\(_\perp\) a is the average value of uniaxial compressive strength in the parallel joint direction.

Figure 7: Comparison of UCS calculated by various empirical models.

low strength, the performance of the needle penetrometer is better.

5. Estimation of Mechanical Parameters of Sericite Schist

5.1. Estimation of Parameters (c, \(\varphi\), \(E_m\)) Based on Hoek-Brown Criterion

5.1.1. Theoretical Analysis. Due to the particularity of tunnel soft surrounding rock, we consider the influence of rock mass structure characteristics on the mechanical parameters such as \(c\) and \(\varphi\) of sericite schist. Then, use the generalized Hoek-Brown criterion [26] as the estimation basis. The formulas are as follows:

\[
\sigma_1 = \sigma_3 + c_i \left( \frac{m_b}{c_i} + s \right) \sigma_3 \theta ,
\]

\[
m_b = m_1 \exp \left( \frac{GSI - 100}{28 - 14D} \right) ,
\]

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

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

where \(\sigma_1\) is the maximum effective principal stress of rock mass failure, \(\sigma_3\) is the minimum effective principal stress of rock mass failure, \(c_i\) is the uniaxial compressive strength of intact rock, \(m_b\) and \(s\) are the material parameters of rock mass, \(a\) is the constant representing jointed rock mass, \(m_1\) is the material parameter of rock mass, GSI is the geomechanical strength index, and \(D\) is the disturbance coefficient of rock mass.

Some parameters in the Mohr-Coulomb strength criterion can be expressed equivalently by the Hoek-Brown criterion. The expressions of cohesion \(c\), internal friction angle \(\varphi\), and elastic modulus \(E_m\) are shown in the following formulas:

\[
\sigma_{3\text{max}} = \frac{\sigma_3}{c_i} ,
\]

\[
\sigma_{3\text{max}} \frac{\sigma_{cm}}{\gamma H} = 0.47 \left( \frac{\sigma_{cm}}{\gamma H} \right)^{-0.94} \text{tunnel},
\]

\[
\sigma_{cm} = c_i \frac{m_b + 4s - a(m_b - 8s)}{2(1 + a)(2 + a)} ,
\]

\[
c = \frac{c_i[(1 + 2a)s + (1 - a)m_b \cdot \sigma_{3\text{a}} \cdot (s + m_b \cdot \sigma_{3\text{a}})^{a-1}]}{(1 + a)(2 + a) \sqrt{1 + [6a \cdot m_b (s + m_b \cdot \sigma_{3\text{a}})^{a-1}]/(1 + a)(2 + a)}} ,
\]

\[
\varphi = \sin^{-1} \left[ \frac{6a \cdot m_b (s + m_b \cdot \sigma_{3\text{a}})^{a-1}}{2(1 + a)(2 + a) + 6a \cdot m_b (s + m_b \cdot \sigma_{3\text{a}})^{a-1}} \right] ,
\]

\[
E_m = \left( 1 - \frac{D}{2} \right) \sqrt{\frac{c_i}{100 \cdot 10^{(GSI-10)/40}}} \sigma_{ci} \leq 100 \text{ MPa} ,
\]
where $\sigma_{3\text{max}}$ is the upper limit of the minimum principal stress of rock mass, $\sigma_{cm}$ is the uniaxial compressive strength of rock mass, $\gamma$ is the bulk density of rock mass, and $H$ is the buried depth of the tunnel.

### 5.1.2. Estimation Results

Using the methods and experience tables in references [27, 28], we determine the parameters such as $m_b$, $s$, and GSI of the excavated surrounding rock (as shown in Figure 8). The estimated results are shown in Table 4.

As can be seen from Table 3, the $c$ value estimated by Hoek-Brown criterion in the test section is between 0.044 and 0.251 MPa, the $\varphi$ value is between 10.88 and 24.11°, and the $E_m$ is between 0.088 and 0.292GPa.

### Table 3: Equations related to UCS and NPI found in previous studies [8–10, 14, 17–25].

| Authors                  | Function                               | Rock types                                                                 |
|--------------------------|----------------------------------------|---------------------------------------------------------------------------|
| Okada et al. (1985) [17] | log UCS = 0.978 log NPI + 1.599        | Rocks and soilcrete                                                        |
| Takahashi et al. (1988)  | UCS = 1.5395 $\log^{0.9896}$           | Sandstone, mudstone, conglomerate, greywacke, and tuff samples            |
| Yoshikazu et al. (1997)  | log UCS = 0.982 log NPI – 0.209        | Pyroclastic rocks                                                          |
| Uchida et al. (2004) [20]| UCS = 27.3NPI + 132                    | Sandstone                                                                  |
| Erguler and Ulusay. (2007, 2009) [21]| UCS = 0.51NPI$^{0.8575}$            | Marble, siltstone, shale, and tuff                                        |
| Ulusay and Erguler. (2012) [22]| UCS = 0.402NPI$^{0.929}$            | Marl, tuff, mudstone, siltstone, sandstone, tough clay, and greywacke      |
| Aydan. (2012) [23]       | UCS = 0.2NPI                           | Tuff, sandstone, pumice, marl, limestone, lignite, mudstone, siltstone,    |
| Aydan and Ulusay. (2013) [24]| UCS = 0.3NPI                        | and loam                                                                   |
| Kahraman et al. (2017)   | UCS = 0.35NPI                          | Turkish tufts                                                              |
| Rahimi et al. (2020) [14]| UCS = 0.49NPI                          | Cayirham coal                                                              |

where $c$, $\varphi$, and $E_m$ are cohesion force (MPa), friction angle (°), and Young’s modulus (GPa), respectively. The above three formulas date from the empirical formulas of many engineering practices, and the suitable rock types are tuff, sandstone, soapstone, pumice, limestone, lignite, marl, and loam.

The estimation results between the penetrometer and Hoek Brown criterion are shown in Figure 9.

According to Figure 7, in the estimation of the needle penetrometer, the average of $c$ is 0.203 MPa, $\varphi$ is 18.224°, $E_i$ is 0.206 GPa. In general, the mechanical parameters of rock mass estimated based on Hoek-Brown criterion and

### Table 4: Sericite schist $c$, $\varphi$, $E_m$ estimation results.

| Section               | $c$ (MPa) | $\varphi$ (°) | $E_m$ (GPa) |
|-----------------------|-----------|---------------|-------------|
| DK163 + 066 ~ 076     | 0.044–0.115 | 10.880–21.865 | 0.088–0.292 |
| DK164 + 84.6 ~ 106    | 0.062–0.152 | 14.667–24.106 | 0.106–0.187 |
| DK164 + 138 ~ 147     | 0.122–0.251 | 14.394–19.439 | 0.146–0.212 |

### 5.2. Needle Penetrometer Estimation

Use the formula in the ISRM-recommended method [29] to estimate

$$c = 0.04 \text{NPI},$$

$$\varphi = 54.9 \left[1 - \exp \left(-\frac{\text{NPI}}{10}\right)\right],$$

$$E_i = 0.05 \text{NPI},$$

where $c$ is the cohesion force (MPa), NPI is the needle penetrating index (N/mm), $\varphi$ is the friction angle (°) of vertical foliation, and $E_i$ is Young’s modulus (GPa). The above three formulas date from the empirical formulas of many engineering practices, and the suitable rock types are tuff, sandstone, soapstone, pumice, limestone, lignite, marl, and loam.

The estimation results between the penetrometer and Hoek Brown criterion are shown in Figure 9.

According to Figure 7, in the estimation of the needle penetrometer, the average of $c$ is 0.203 MPa, $\varphi$ is 18.224°, $E_i$ is 0.206 GPa. In general, the mechanical parameters of rock mass estimated based on Hoek-Brown criterion and
Figure 9: Estimating results of needle penetration apparatus and Hoek-Brown criterion. (a) Estimated results of c. (b) Estimated results of \( \phi \). (c) Estimated results of \( \frac{E_m}{E_i} \).
NPI can be used as a reference. However, in order to quickly obtain the mechanical parameters of sericite schist in the field, the needle penetrometer test is a fast and economic means.

6. Conclusions

In this paper, we analyze the characteristics of large deformation of Baishitou tunnel, take an in situ test, obtain the NPI index of sericite schist, estimate the basic mechanical parameters of Sericite schist, and discuss some methods of soft rock strength test. We can draw the following conclusions:

(1) The characteristics of extrusion rock tunnel are summarized as large deformation, fast deformation rate, and obvious construction disturbance

(2) The reference value of penetration index of sericite schist (the vertical joint direction) is 3.90~7.77 N/mm, and the parallel joint direction is 1.27~2.99 N/mm

(3) The uniaxial compressive strength estimated by a penetrometer is 0.78~8.53 MPa, and the strength of surrounding rock is negatively correlated with the amount of deformation. It can be concluded that the insufficient strength of surrounding rock is the fundamental reason for large deformation

(4) The accuracy of the penetrometer in the estimation of mechanical parameters such as $c$ and $f_{um}$ of sericite schist needs to be further improved. In addition, in order to quickly obtain the mechanical parameters of soft rocks in the field, the needle penetrometer test is a fast and economic means

Data Availability

The data that support the conclusions of this study are available from the text and the corresponding author upon reasonable request.

Conflicts of Interest

There are no conflicts of interest with respect to the results of this paper.

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