Experimental study on the mechanical of deep-buried soft rock under triaxial loading and unloading

Lehua Wang\textsuperscript{1,2}, Guangqiang Feng\textsuperscript{1,2}, Bingyi Zhang\textsuperscript{1,2}, Weizheng Shi\textsuperscript{1,2}, Can Chen\textsuperscript{1,2} and Tianzhu Huang\textsuperscript{1,2*}

\textsuperscript{1} College of Civil Engineering and Architecture, China Three Gorges University, Yichang City, Hubei Province, 443002, China
\textsuperscript{2} Key Laboratory of Geological Hazards on Three Gorges Reservoir Area of Ministry of Education, China Three Gorges University, Yichang City, Hubei Province, 443002, China
\textsuperscript{*Corresponding author’s e-mail: 2016206010@stu.cqust.edu.cn
\textsuperscript{*Corresponding author’s ORCID No: https://orcid.org/0000-0002-0041-2481

Abstract. This study investigated the changes in the mechanical properties of the soft surrounding rocks in the deep-buried cavern during the excavation unloading process. The argillaceous sandstone in the main diversion tunnel of the Panlong pumped storage power station in Chongqing is the case study. Triaxial loading and unloading tests were carried out to discuss rock samples’ strength, deformation, and failure characteristics under different stress paths. The results showed that the soft rock’s peak strength, strain, deformation modulus, and cohesion in the triaxial loading test are greater than those of the unloading test. During the loading process, the soft rock’s elastic modulus and deformation modulus increase with the confining pressure. However, a more significant increase was observed in the deformation modulus. Under loading conditions, the soft rock suffers from double-shearing shear failure with a failure angle of about 65°. Under unloading conditions, single-shearing shear failure is dominant and accompanied by tensile failure. There are many tension cracks on the surface of the rock sample under low confining pressure, and lateral expansion occurs under high confining pressure.

1. Introduction
Soft rock is the most widely distributed rock on the earth’s surface, which is easily weathered and decomposed, therefore, has characteristics of loose crushing and low strength. Some scholars have reported that about two-thirds of all hydropower projects involve soft rock engineering in geological applications. Many soft rock tunnels cause engineering problems such as floor uplift and roof subsidence in deep-buried engineering. This poses a severe challenge to the research of deep soft rock engineering. Huang\textsuperscript{[1]} conducted triaxial unloading tests on sandy mudstone with different unloading rates and stress paths to study soft rock’s dilation and fracture characteristics. He\textsuperscript{[2]} simulated the process of rockburst by the true triaxial unloading test of rapidly unloading the stress in one direction, and found that the failure modes of rockburst were tensile fracture, stepped fracture, and overall stress fracture. Feng\textsuperscript{[3]} carried out a true triaxial unloading test of granite and analyzed granite’s rebound deformation and elastic brittle failure characteristics using acoustic emission information. Li\textsuperscript{[4]} carried out a true triaxial unloading test and found that the aspect ratio and intermediate principal stress of the sample affect the failure mode, peak strength, and failure degree of hard rock. Kong\textsuperscript{[5]} carried out a true triaxial loading test of volcanic rock and reported that the brittleness of rock failure increased with the intermediate principal stress. Malandraki\textsuperscript{[6-7]} and Gutierrez\textsuperscript{[8]} summarized many research results on
the shear yield characteristics of structured soils and proposed a double logarithmic coordinate characterization method of tangential stiffness and axial strain in the shear test. Du\textsuperscript{[9-11]} carried out a true triaxial loading and unloading tests and reported that the lithology, size, and intermediate principal stress of rock specimens all significantly affect the failure characteristics and peak strength of the specimens. Jiang\textsuperscript{[12]} studied the mechanical characteristics and strength parameters of gray sandstone under different stress paths by carrying out triaxial loading tests. Zhao\textsuperscript{[13]} studied deep-buried granites’ deformation and failure characteristics under high confining pressure cyclic loading and unloading by applying cyclic loading and unloading tests under different confining pressures. Zhou\textsuperscript{[14]} conducted triaxial tests on weathered granite soil under different unloading and reloading times, revealing the influence of unloading and reloading paths on the mechanical properties of material strength characteristics, deformation modulus, etc. Martin\textsuperscript{[15-16]} analyzed the results of many granite mechanical properties and proposed a Cohesion Weakening Friction Strengthening (CWFS) model suitable for deep rocks. Li\textsuperscript{[17]} studied the stress-strain and failure characteristics of sandstone under unloading stress. Rong\textsuperscript{[18]} carried out a true triaxial disturbance unloading test of a rock to analyze the evolution law of rock mechanical parameters under different stress paths.

In summary, there are many studies on rock unloading, but there are few studies on deep-buried soft rocks. Therefore, this study investigated the argillaceous sandstone’s deformation, strength, and failure characteristics under triaxial loading and unloading tests. The research conclusions can provide references for understanding the mechanical properties of soft rock surrounding rock during the construction of deep underground caverns.

2. Test plan and identification of rock samples

2.1. Preparation and screening

The rock samples are argillaceous sandstone, the surrounding rock of the main diversion tunnel of Chongqing Panlong pumped storage power station. First, prepare a standard cylindrical test sample with a diameter of 50 mm, a height of 100 mm, and an end flatness \(\leq \pm 0.5 \text{ mm}\) (Figure 1).

![Figure 1. Standard samples](image1)

![Figure 2. Ultrasonic testing instrument](image2)

![Figure 3. The physical properties of rock samples](image3)
The physical properties of rock samples are measured by an Ultrasonic testing instrument (Figure 2) and balance. The wave velocity and density of rock samples are shown in Figure 3. Rock samples with similar physical properties were selected for the test. The wave velocity range of rock samples is determined to be $2.90\text{–}2.95\text{Km/s}$, and the density is $2.63\text{g/cm}^3$.

2.2. Identification of rock samples

AB204-S Analytical Balance (EX001) and TAS-986G Atomic absorption spectrophotometer (EX006) were used to detect the chemical constituents. The test results are shown in Table 1. After slicing the rock sample, the mineral composition was identified by the OLYMPUS BX51 microscope. The content of each mineral component is shown in Table 2. The chemical and mineral composition test results showed that the sample has a fine-silt structure, and the cementation form is a base porous composite cementation.

### Table 1. Chemical composition and content of samples

| Chemical composition | SiO$_2$ | MgO | Fe$_2$O$_3$ | Al$_2$O$_3$ | TiO$_2$ | CaO | K$_2$O | Na$_2$O |
|----------------------|---------|-----|------------|------------|---------|-----|-------|-------|
| Content /%           | 60.54   | 1.83| 3.46       | 9.11       | 0.74    | 10.66| 2.23  | 0.86  |

### Table 2. Mineral composition and content of samples

| Mineral | Detrital composition | Cement composition |
|---------|----------------------|--------------------|
|         | Quartz                    | Mica               |
|         | Rock fragments           | Calcite            |
|         | Iron                      | Plagioclase        |
|         | Clay minerals            | Calcite            |
| Content /% | 42 | 8 | 9 | 14 | 2 | 3 | 18 | 10 |

2.3. Test plan

In this study, the triaxial loading test and triaxial unloading test of argillaceous sandstone were carried out. The maximum buried depth of the tunnel of Chongqing Panlong pumped storage power station is about 500 m, which is a deep-buried environment with a maximum in-situ stress level of about 15MPa. Therefore, to truly ascertain the actual situation of the power station, the confining pressure of 5, 10, and 15 are selected.

(1) Triaxial loading test

The stress path of the triaxial loading test is shown in Figure 4, with two stages. The first is the hydrostatic pressure loading stage, with an axial pressure (0.05 MPa/s) and confining pressure (0.05 MPa/s) applied to the design values (5 MPa, 10 MPa, 15 MPa). The second is the axial loading failure stage, with a constant confining pressure, and load the axial pressure at a rate of 0.005 mm/s until the sample fails.

(2) Triaxial unloading test

The triaxial unloading test was carried out with constant axial pressure $\sigma_1$ and unloading confining pressure $\sigma_3$. The axial pressure $\sigma_1$ is 80% and 90% of the peak triaxial strength $\sigma_c$ under different confining pressures. The stress path of the triaxial unloading test is shown in Figure 4, and the test was divided into three stages. The first is the same as the triaxial loading test. The second is the axial pressure loading stage, where the axial force is loaded to the set value at a rate of 0.25 MPa/s. The last is the confining pressure unloading stage, where a constant axial pressure was maintained, and unloading the confining pressure at a rate of 0.005MPa/s until the rock sample fails.
3. Study on mechanical characteristics of triaxial loading test

3.1. Stress-strain curve analysis of triaxial loading

Triaxial loading tests of the soft rock with different confining pressures (5, 10, 15 MPa) were carried out. According to the test data, the stress-strain curve is shown in Figure 5, and the strength parameters of soft rock under different confining pressures are shown in Table 3.

![Stress-strain curve under triaxial loading](image)

**Figure 5.** Stress-strain curve under triaxial loading

**Table 3.** Strength parameters of triaxial loading

| $\sigma_3$/MPa | Peak strength $(\sigma_1 - \sigma_3)$ /MPa | Residual strength /MPa |
|---------------|----------------------------------------|------------------------|
| 0             | 46.52                                  | —                      |
| 5             | 69.44                                  | 34.18                  |
| 10            | 89.13                                  | 42.89                  |
| 15            | 105.59                                 | 53.39                  |

From Figure 5 and Table 3, the peak strength of soft rock increases significantly with the confining pressure. The reason is that with the increase of confining pressure, the friction between fracture surfaces in rock mass increases, inhibiting the generation of shear slip. Therefore, the confining pressure can enhance the strength of soft rock. In addition, the residual strength of soft rock increases with the confining pressure. The higher the confining pressure, the greater the deformation of soft rock before failure. Under low confining pressure, the soft rock has a ductile failure characteristic after reaching peak strength. However, when the confining pressure reached 15 MPa, the stress-strain curve dropped sharply, depicting that the soft rock has the characteristics of a brittle failure. The average
uniaxial compressive strength of rock samples is 46.52 MPa, ranging from 25 to 50MPa. Therefore, the rock sample belongs to a class of high-stress soft rocks. According to its mineral composition, the quartz content is high with calcareous cementation.

3.2. Strength characteristic analysis

The relationship between peak strength and confining pressure is shown in Figure 6. The confining pressure and the peak strength rock have a linear relationship when the confining pressure is within 15MPa. The fitting function of peak strength $\sigma_c$ and confining pressure $\sigma_3$ is obtained by fitting:

$$\sigma_c = 47.635 + 3.838\sigma_3, R^2 = 0.99$$

(Figure 6. Relationship between Peak strength and confining pressure)

According to the Mohr-Coulomb strength criterion, a shear failure plane is usually formed when rock is subjected to three-dimensional stress, and the shear stress on the shear failure plane is the shear strength of rock. The Mohr-Coulomb shear strength formula is

$$\tau = c + \sigma \tan \phi$$

Where: $c$ is cohesion, $\phi$ is the friction angle, $\sigma$ is the normal stress on the shear failure plane. Among them, $c$ and $\phi$ are two important indexes of rock shear strength parameters. Equation (2) is expressed by principal stress as follows:

$$\sigma_1 = \frac{2c \cos \phi}{1 - \sin \phi} - \frac{1 + \sin \phi}{1 - \sin \phi} \sigma_3$$

(Fitting the experimental data can get the relationship between $\sigma_1$ and $\sigma_3$:)

$$\sigma_1 = K \sigma_3 + Q$$

Where: The K and Q are the undetermined constant obtained by linear fitting. It can be obtained from Equation (3) as follows:

$$K = \tan^2 \alpha = \tan^2 (45^0 + \phi / 2)$$

$$Q = \frac{2c \cos \phi}{1 - \sin \phi} = 2c \tan \alpha$$

According to the test results, the values of K and Q are calculated and substituted into (5) and (6) to compute the cohesion, $c$ of the rock sample found to be 12.16MPa, with an internal friction angle $\phi$ of 35.92°.

3.3. Deformation characteristics analysis

The deformation characteristics of rock are usually expressed by Young’s modulus, Peak strain, and Poisson’s ratio. Young’s modulus of rock mainly includes tangent modulus, secant modulus, and average modulus. The secant modulus, also known as deformation modulus, is generally used as the stress-strain ratio at half of the peak strength of rock. For the unloading stage, the ratio of stress to
strain is the deformation modulus. The average modulus, also known as the elastic modulus, is the slope of the approximate straight line of the stress-strain curve, as shown in Figure 7.

![Figure 7. Elastic modulus (left) and deformation modulus (right)](image)

Through the stress-strain curve, the elastic and deformation moduli of rock samples under different confining pressures in the loading stage are calculated, as shown in Table 4.

| σ₃/MPa | E/GPa | Growth rate of E/% | Deformation modulus / GPa | Growth rate of Deformation modulus/ % |
|--------|-------|--------------------|---------------------------|--------------------------------------|
| 0      | 6.60  | —                  | —                         | —                                    |
| 5      | 10.40 | 57.58              | 8.84                      | 58.71                                |
| 10     | 12.04 | 82.42              | 10.98                     | 97.13                                |
| 15     | 14.78 | 123.94             | 14.83                     | 166.25                               |

Note: the growth rate is (modulus under certain confining pressure - modulus under uniaxial condition) / modulus under uniaxial condition × 100%.

From Table 4, the elastic modulus is greater than the deformation modulus under the same confining pressure. This is because the deformation modulus is the ratio of stress to strain when the peak strength is half, including the compaction stage. The elastic modulus is the ratio of stress to strain in the approximate straight line. The elastic and deformation moduli of soft rock increase with the confining pressure; however, the deformation modulus increased more significantly.

![Figure 8. Relationship between elastic modulus and confining pressure](image)  
![Figure 9. Relationship between deformation modulus and confining pressure](image)

To analyze the relationship between deformation parameters and confining pressure, linear fitting and polynomial fitting is carried out, respectively, as shown in Figure 7 and Figure 8.

The linear fitting of the elastic modulus:

\[ E = 7.028 + 0.5236\sigma₃, R^2 = 0.965 \quad (7) \]

The polynomial fitting is
The linear fitting of the deformation modulus:

\[ E_d = 5.567 + 0.5984\sigma_3, R^2 = 0.984 \]  

(9)

The polynomial fitting is

\[ E_d = 5.712 + 0.5114\sigma_3 + 0.0058\sigma_3^2, R^2 = 0.973 \]  

(10)

Where \( E \) represents the modulus of elasticity, \( E_d \) represents the modulus of deformation, and \( \sigma_3 \) represents the confining pressure.

Comparing the two fitting results, the \( R^2 \) of the linear fitting is higher than that of the polynomial fitting, indicating that the linear fitting effect is better. That is, the elastic and deformation moduli of the rock sample increase linearly with the confining pressure.

3.4. Failure characteristics analysis

The failure characteristics of rock samples under uniaxial and triaxial loading under different confining pressures are shown in Figure 10 and Figure 11. The right side of Figure 10 and Figure 11 shows the distribution of cracks on the cylindrical rock sample surface after failure. The purpose is to clearly show the fracture development and failure angle of rock samples.

From Figure 10, the failure form of soft rock under uniaxial compression is X-shaped conjugate shear failure, and the failure angle is smaller than that under triaxial loading. During the uniaxial compression test, there was a clear popping sound accompanied by flying debris.

It can be seen from Figure 11 that the failure angle formed by triaxial loading of soft rock is about 65°, which is close to the failure angle \( (45° + \varphi/2) \) calculated by the Mohr-Coulomb criterion. The failure form of triaxial loading is mainly a shear failure, and the cracks generated on the failure surface are narrow. The failure surfaces coincide well, indicating that the confining pressure has a limit on the failure of soft rock. With the increase of confining pressure, the failure surface is transformed from two ends to one end.

By comparing the uniaxial compression and triaxial loading, the fracture surface of uniaxial compression is coarser than that of triaxial loading. As a result, falling blocks were observed between fractured surfaces. Furthermore, there are many secondary cracks around the fracture surface, depicting that the failure of the rock sample is more severe under unconfined action. In addition, the failure angle under uniaxial compression is smaller than that under triaxial loading.

![Figure 10. Failure characteristic of uniaxial compression](image)

(a) Confining pressure 5MPa
4. Study on mechanical characteristics of triaxial unloading test

4.1. Stress-strain curve analysis of triaxial unloading

Triaxial unloading tests of soft rock with different confining pressure levels (5, 10, 15MPa) were carried out. When the axial pressure $\sigma_1$ is 80% of the peak triaxial strength $\sigma_c$, the unloading stress-strain curve of soft rock is shown in Figure 12.

![Stress-strain curve under triaxial unloading](image)

**Figure 12.** Stress-strain curve under triaxial unloading

It can be concluded from Figure 12 that the stress difference ($\sigma_1-\sigma_3$) and strain show an excellent linear growth relationship in the loading stage. In the unloading confining pressure stage, the stress difference is almost constant, while the deformation increases rapidly, and the stress-strain curve showed a plateau and then suddenly failed. In the loading stage, the slope of the curve increased with the confining pressure. This is because the confining pressure puts the rock sample in a three-dimensional stress state, improving the strength. The ability of the soft rock to resist deformation in the unloading stage is less than that in the loading stage, indicating that unloading easily causes severe rock failure.

4.2. Strength characteristic analysis

The strength parameters obtained from triaxial unloading test under different stress levels are shown in Table 5.
Table 5. Strength parameters of triaxial unloading

| Axial stress level | $\sigma_3$ /MPa | $\sigma_1$ /MPa | $\sigma_3^*$ /MPa | Peak strength ($\sigma_1,\sigma_3^*$)/MPa |
|-------------------|----------------|----------------|------------------|----------------------------------------|
| 80%$\sigma_c$     | 5             | 56.0           | 1.2              | 54.8                                   |
|                   | 10            | 79.0           | 3.0              | 76.0                                   |
|                   | 15            | 95.0           | 7.5              | 87.5                                   |
|                   | 5             | 63.0           | 4.2              | 58.8                                   |
| 90%$\sigma_c$     | 10            | 90.0           | 6.5              | 83.8                                   |
|                   | 15            | 108.0          | 11.5             | 96.5                                   |

It can be concluded from Table 5 that the peak strength of rock samples under triaxial unloading increases with the increase of confining pressures. This is similar to triaxial loading, but the strength is generally lower than that of triaxial loading, showing that the strength of the rock samples under triaxial unloading is lower. Different axial stress levels have great influence on the unloading failure of rock. When the axial stress level is 80% $\sigma_c$, a large amount of unloading is required to cause the rock samples to fail. When the axial stress level increases to 90% $\sigma_c$, small unloading can make the rock samples fail. According to the Mohr-Coulomb strength criterion, the cohesion of rock samples is 9.76 MPa, and the internal friction angle is 46.52° under triaxial unloading.

4.3. Deformation characteristics analysis

To study the deformation and failure law of rock samples in the unloading process, the unloading value $\Delta\sigma_3$ is introduced for quantitative analysis. The unloading value $\Delta\sigma_3$ represents the percentage of the unloading value at the current stage in the total unloading confining pressure, as shown in the following formula:

$$\Delta\sigma_3 = \frac{\sigma_3 - \sigma_3^i}{\sigma_3 - \sigma_3^*} \times 100\%$$  \hspace{1cm} (11)

Where $\sigma_3$ represents the design confining pressure value, $\sigma_3^i$ represents the confining pressure value at the $i$-th moment of the confining pressure unloading stage, and $\sigma_3^*$ represents the confining pressure value at the time of unloading failure. Taking the triaxial unloading test with 80% $\sigma_c$ axial stress as an example, the degradation law of deformation modulus during unloading failure under different confining pressures is analyzed. The unloading value is divided into seven levels: 0%, 20%, 40%, 60%, 80%, 90%, and 100%. Zero percent (0%) of the unloading value is the deformation modulus at the beginning of unloading, and 100% of the unloading value is the deformation modulus at the failure of the rock samples. Based on the test data, the change of deformation modulus of rock samples under different confining pressures and unloading degrees is calculated, as shown in Table 6. The relationship between deformation modulus and unloading value under different confining pressures is shown in Figure 13. The relationship between average decline and unloading value is shown in Figure 14.

According to Table 6 and Figure 14, the unloading stage is divided into four stages: initial unloading stage, middle unloading stage, late unloading stage, and final unloading stage, representing 0–20%, 20–60%, 60–90%, and 90–100% of unloading value, respectively. To fully show the change rule of the average reduction range of 0–90%, the 90–100% case is not shown in Figure 14. The results are as follows: (1) From Figure 13, the deformation modulus shows a decreasing trend during unloading under different confining pressures. The decreasing rate is low at the initial, middle, and late stages of unloading, but it drops sharply at the final stage. (2) The average decrease of the deformation modulus first decreases and then increases with the unloading value. The reason is that the rock sample has just begun to crack at the initial unloading stage, with the ability of the rock sample to resist deformation reduced. In the middle stage of unloading, the micro-cracks in the rock sample did not develop and penetrate, so the average decrease in deformation modulus declined. In the later stage of unloading, the micro-cracks in the rock sample gradually developed. As a result, the rock sample began to expand laterally, and the decline was higher than that in the initial stage. The increase of strain will be
significantly greater than that of the stress. Therefore, the deformation modulus of rock samples will dramatically decrease compared with the initial stage of unloading. When entering the final stage of unloading, the internal cracks of the rock sample are entirely penetrated, and the deformation modulus drops sharply. Therefore, the decline in the final deformation modulus is the highest. (3) When the unloading value is constant, the average decline of deformation modulus increases with confining pressure. It shows that the rock sample is easier to be destroyed when unloading under high confining pressure.

### Table 6. Deformation modulus changing with unloading value during triaxial unloading

| $\sigma_3$/MPa | Deformation modulus/GPa | 0%  | 20%  | 40%  | 60%  | 80%  | 90%  | 100% |
|---------------|------------------------|-----|------|------|------|------|------|------|
| 5             |                        | 6.16| 5.99 | 5.89 | 5.79 | 5.67 | 5.57 | 2.96 |
|               | Deformation modulus/GPa |     |      |      |      |      |      |      |
|               | Decrease rate/%        |     | 2.83 | 1.69 | 1.70 | 1.94 | 1.78 | 46.97|
|               | Average decline/%      |     | 14.13| 8.44 | 8.50 | 9.68 | 17.80| 469.67|
| 10            |                        | 9.97| 9.58 | 9.34 | 9.06 | 8.66 | 8.35 | 4.07 |
|               | Deformation modulus/GPa |     |      |      |      |      |      |      |
|               | Decrease rate/%        |     | 3.95 | 2.45 | 2.99 | 4.39 | 3.66 | 51.26|
|               | Average decline/%      |     | 19.76| 12.27| 14.94| 21.96| 36.59| 512.64|
| 15            |                        | 13.15| 12.65| 12.36| 12.03| 11.57| 11.26| 5.00 |
|               | Deformation modulus/GPa |     |      |      |      |      |      |      |
|               | Decrease rate/%        |     | 3.79 | 2.29 | 2.64 | 3.82 | 2.68 | 55.66|
|               | Average decline/%      |     | 18.94| 11.43| 13.19| 13.39| 26.78| 556.55|

Note: the average decline is the decline rate of this stage / (unloading value of this stage - unloading value of the previous stage) × 100%

**Figure 13.** Relationship between deformation modulus and unloading value

**Figure 14.** Relationship between the average decline and unloading value

### 4.4. Failure characteristics analysis

The failure characteristics of triaxial unloading rock samples under different confining pressures are shown in Figure 15.
11th Conference of Asian Rock Mechanics Society

IOP Conf. Series: Earth and Environmental Science 861 (2021) 022066
doi:10.1088/1755-1315/861/2/022066

From Figure 15(a), the failure mode of the rock sample at a confining pressure of 5MPa is dominated by shear. There are more cracks and tensile failure in the vicinity of the primary shear plane, with a higher failure angle. Due to the Poisson effect, the rock sample generates tensile stress in the transverse direction under axial force. As a result, many tensile cracks on the surface gradually extended and penetrated both ends, causing the rock sample to be destroyed.

Figure 15(b)–(c) shows that the rock sample’s lateral deformation is relatively large when the confining pressure is 10MPa, with a significant expansion phenomenon. The rock sample is mainly sheared when the confining pressure is 15MPa, and a few large tensile cracks appear near the main shear surface. The surface of the rock sample is relatively clean, and the shear plane runs through the top and bottom of the rock sample.

Comparing the failure modes of rock samples under triaxial loading and triaxial unloading, when the confining pressure increased, the penetration of the failure surface exhibited the opposite law, indicating that the confining pressure has a greater impact on the failure of the rock sample. The rock sample shows a double shear plane failure in the triaxial loading test, while the triaxial unloading test shows a single shear plane failure. In addition, the fracture surface of the rock sample under unloading conditions is wide, with powder-like particle dispersion, while the fracture surface under compression conditions is relatively small.

5. Conclusion

(1) Under triaxial loading, the peak strength, peak strain, elastic modulus, and deformation modulus increase linearly with confining pressure. Combined with the Mohr-Coulomb strength criterion, the cohesion of the rock sample is 12.16 MPa and the internal friction angle is 35.92° under the triaxial loading condition. In addition, the cohesion is 9.76 MPa, and the internal friction angle is 46.52° under triaxial unloading conditions.

(2) The peak strength, peak strain, deformation modulus, and cohesion of soft rock under the triaxial unloading test are lower than those under the triaxial loading test, with a small difference in elastic modulus. During the unloading process, the deformation modulus decreases gradually. The decrease is not obvious at the initial, middle and late stages of unloading but sharply at the end.

(3) The rock failure mode under uniaxial compression is X-shaped conjugate shear failure with a small failure angle. Under triaxial loading, the failure mode is a double-plane shear failure, and the failure angle is about 65°. The triaxial unloading failure mode is mainly single-plane shear failure, accompanied by tension failure. There are many tension cracks on the surface of rock samples under
low confining pressure, where lateral dilation occurred under high confining pressure. The failure surface of the rock sample is wide, with a lot of powders in the cross-section, and the rock sample has an obvious dilation phenomenon.

Acknowledgments
China National Natural Science Foundation Joint Funded Project (No.U1965109); Hubei Provincial Natural Science Foundation Innovation Group Project (No.2020CFA049).

References
[1] Huang X, Liu Q S and Liu B 2017 Experimental study on the dilatancy and fracturing behavior of soft rock under unloading conditions Int. J. Civ. Eng. 15 921-948
[2] HE M C, ZHAO F and CAI M 2015 A novel experimental technique to simulate pillar burst in laboratory Rock Mech. Rock Eng. 48 1833-1848
[3] Feng G L, Feng X T and Chen B R 2019 Effects of structural planes on themicroseismicity associated with rockburst development processes in deep tunnels of the Jinping-II Hydropower Station China Tunn. Undergr. Space Technol. 84 273-280
[4] Li X B, Feng F and Li D Y 2018 Failure characteristics of granite influenced by sample height-to-width ratios and intermediate principal stress under true-triaxial unloading conditions Rock Mech. Rock Eng. 51 1321-1345
[5] Kong R, Tuncay E and Ulusay R 2021 An experimental investigation on stress-induced cracking mechanisms of a volcanic rock Eng. Geol. 280 1-11
[6] Malandraki V and Toll D G 1996 The definition of yield for bonded materials Geotech. Geol. Eng. 14 67-82
[7] Malandraki V and Toll D G 2000 Drained probing triaxial tests on a weakly bonded artificial soil Geotechnique 50 141-151
[8] Gutierrez M, Nygard R and Hoeg K 2008 Normalized undrained shear strength of clay shales Eng. Geol. 99 31-39
[9] Du K, Tao M and Li X B 2016 Experimental study of slabbing and rockburst induced by true-triaxial unloading and local dynamic disturbance Rock Mech. Rock Eng. 49 1-17
[10] Li X B, Feng F and Li D Y 2018 Failure characteristics of granite influenced by sample height-to-width ratios and intermediate principal stress under true-triaxial unloading conditions Rock Mech. Rock Eng. 51 1321-1345
[11] Du K, Li X B and Li D Y 2015 Failure properties of rocks in true triaxial unloading compressive test Trans Nonferrous Met Soc China 25 571-581
[12] Jiang Y, Zhou H, Lu J J, Zhang C Q, Gao Y and Chen Q 2019 Study on mechanical properties and strength parameters of gray sandstone under different stress paths Chin. J. Rock Mech. Eng. 38 815-824
[13] Zhao J, Guo G T, Xu D P, Huang X, Hu S, Xia Y L and Zhang D 2020 Experimental study of deformation and failure characteristics of deeply-buried hard rock under triaxial and cyclic loading and unloading stress paths Rock Soil Mech 41 1521-1530
[14] Zhao Y R, Yang H Q and Huang L P 2019 Mechanical behavior of intact completely decomposed granite soils along multi-stage loading–unloading path Eng. Geol. 260 1-8
[15] Martin C D and Chandler N 1994 The progressive fracture of Lac du Bonnet granite Int. J. Rock Mech. Min. Sci. & Geomech. Abstr. 31 643-659
[16] Martin C D 1997 Seventeenth Canadian geotechnical colloquium: the effect of cohesion loss and stress path on brittle rock strength Can Geotech J 34 698-725

[17] Li J L, Wang R H, Jiang Y Z, Liu J and Chen X 2010 Experimental study of sandstone mechanical properties by unloading triaxial tests Chin. J. Rock Mech. Eng. 29 2034-2041

[18] Rong H Y, Li G C and Zhao G M 2020 True triaxial test study on mechanical properties of deep rock mass in different stress paths J. China Coal Soc. 45 3140-3149