The field survey and deformation characteristics of exit slope of Qingshuigou tunnel in the southwest of China

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Received: 13 May 2021 / Accepted: 17 May 2022 / Published online: 31 May 2022
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Abstract
The stability of the slope under excavation in various engineering is a hot issue, and both engineering disturbances and other internal factors have contributed to the instability of the slope. The study focused on the deformation characteristics mechanism of exit slope of Qingshuigou tunnel in Jiasha Town, Gejiu City, Honghe Prefecture, Yunnan, China. Field survey and monitoring are performed to reveal deformation mechanism of the exit slope, which is associated with hydrogeologic conditions, natural weathering, and excavation activities. The exit slope at the interface of the overburdened layer and bedrock has the possibility of landslide through the joint force of rainfall and artificial construction. According to the deformation characteristics of the exit slope and monitoring analysis, the results indicate that the surface displacement at the back scarp of the exit slope is larger than that at the head scarp of the exit slope. The deep displacement at the back scarp is 2–4 times of that at the head scarp of the slope. Based on the geophysical analysis, the overburdened layer mostly has low resistivity. The deformation mechanism of the exit slope is revealed ultimately based on the combined influences of hydrogeologic conditions, natural weathering, and excavation activities. Based on the deformation characteristics and mechanism of the exit slope, comprehensive stabilization treatment measures are proposed to prevent the exit slope from further deformation.

Keywords Exit slope · Excavation · Dangerous rock mass · Deformation characteristics · Reinforcement

Introduction
The instability of slope engineering is an unexpected geological disaster in a mountainous area, which often causes casualties and infrastructure damage, also results in huge economic losses and many potential secondary disasters to the society (Ma et al. 2019, 2022; Wang et al. 2019; Wen et al. 2022b). For example, a total of 184,242 geological disasters occurred during 2007–2019, including 117,086 landslides and 38,632 collapses, accounting for 84.5%, resulting in thousands of deaths and billions of dollars in economic losses (Tang et al. 2019; Wen et al. 2019, 2020a). The deformation and failure mechanism of slope geological disaster is extremely complex and elusive (Wen et al. 2022a). Therefore, it is of great significance to analyze the deformation characteristics of the slope and evaluate its stability.

Recently, an increasing number of scholars have proposed that various internal and external factors should be responsible for the deformation mechanism of rock slopes for geotechnical engineering (Chigira et al. 2010; Gorum et al. 2011). Regrading internal factors, including weak interlayers, types of rock and soil mass, geologic structure,
and geomorphological type, some evidence of these joint- 

influences to the stability of the slope has already been vis-

ible (Tang et al. 2015; Regmi et al. 2017; Gu et al. 2017; Yu 

et al. 2020). Regarding external factors, including rainfall, 
groundwater, reservoir water, and human engineering activi-
ties, they are the main factors triggering slope geological 
disasters (Lee and Pradhan 2007; Stähli et al. 2015; Cogan 
et al. 2018; Tang et al. 2019; Li et al. 2019).

In the mountain area, the high and steep slopes is often triggered by rainfall, human engineer-
ing activities, and so on, which degrades the properties of 
rock and soil mass in physical, chemical, and mechanical 
(Li et al. 2017). Slope unloading is a common geological 
problem, which brings about the rapid adjustment of the 
slope stress field and the gradual deformation of rock masses 
and soil masses (Wen et al. 2021). In this case, pre-existing 
cracks can be dilated, and new cracks can be generated. 
These phenomena are not conducive to the stability of the 
slope (Li et al. 2020; Du et al. 2022). Furthermore, consider-
able attempts to evaluate the stability of the rock slope have 
had a certain process. The stability of the high and steep rock 
slopes forms very important engineering geological issues 
for slope engineering of road and bridge, large scale water 
conservancy, and hydropower projects (Lin et al. 2018; Shi 
et al. 2020). In order to reduce the risk of disasters, a moni-
toring system is an effective way. Furthermore, the stability 
control of the slopes plays an increasingly significant role in 
the success or failure in various geotechnical engineering.

In the southwest of China, there are complex geological 
environment and complex regional internal dynamic and 
extent dynamic geological processes; therefore, slope geo-
logical disasters are prone to occur. The study aims to reveal 
the deformation mechanism of the tunnel exit slope in the 
southwest of China and to put forward the measurements 
for preventing the instability of the slope. Field survey of 
the tunnel exit slope is firstly performed briefly. Afterwards, 
several monitoring methods, such as on-site monitoring and 
geophysical prospecting, have been applied to the exit slope. 
Subsequently, the deformation mechanism of the exit slope 
is revealed based on several factors. Finally, to prevent the 
exit slope from further deformation, comprehensive stabi-
лизation treatment measures are proposed.

**Study area**

**Location of the study site**

The exit slope of Qingshuigou tunnel is located in Jiasha 

Town, Gejiu City, Honghe Prefecture, Yunnan Province, 
China, which is 26 km away from Gejiu City. The exit of 
Qingshuigou tunnel is connected to Yashadi No.1 Bridge, 
as shown in Fig. 1. There are two tunnel exits in the slope,
results, the depth of the strongly weathered rock mass at the upper part of the slope is more than 20 m, and the local part is more than 25 m. The strongly weathered rock mass is broken and in a fragmentary structure. Therefore, the rock mass quality of the slope is poor, and the upper slope is easy to collapse after excavation. At fact, the upper slope at the tunnel exit has collapsed at different elevations due to the influence of the excavation of the road.

**Laboratory tests**

The sampling site is near the study zone, and the rock samples include argillaceous siltstone, quartz sandstone, limestone, and granite. These rock samples are made into standard samples according to the relevant regulation. To ensure the accuracy of the laboratory tests, the rock samples should be kept in their geometric configuration during the preparation. The two ends of the rock samples need to be smooth to ensure that the testing machine can apply pressure to the rock samples evenly. Sonic wave tests are conducted to get rid of irregular rock samples. Based on the sonic wave tests, the $P$ wave velocity of quartz sandstone is determined to be 4778 m/s and the mean $P$ wave velocity of argillaceous siltstone is determined to be 4802 m/s. To determine the uniaxial compressive strength (UCS) of these rock samples, the uniaxial compression tests under saturated conditions are performed using the testing machine, as shown in Fig. 4. Furthermore, to obtain the shear strength of argillaceous siltstone, direct shear tests are conducted with a constant shearing rate.
Table 1 illustrates the physical and mechanical parameters of limestone and granite, including sample depth, natural density, and the UCS. The rock samples are taken from different boreholes. For the CXM1DQZK3, CJS1DQZK ZK4, and CJS1DQZK ZK5 boreholes, the natural density of the limestones increases with the increase of...
of the sample depth, while it decreases broadly with the increase of the sample depth. For the CXM1DQZK3 and CJS1DQZK ZK4 boreholes, the UCS generally decreases with the sample depth, while for the CJS1DQZK ZK5 and CXM1SDZK01 boreholes, the UCS generally increases with the sample depth. For the granite, the natural density varies little with the sample depth for different boreholes. For the CXL2#DQZK2 borehole, the UCS increases gradually with the sample depth, nevertheless, decreases for the CXL2#DQZK2K4 and CXL2DQ2K5 boreholes. On the whole, the strength of the granite is 2–3 times that of the limestone.

Table 2 illustrates the physical and mechanical parameters determined from a series of compression tests for the exit slope.

| Rock type Sample depth (m) | Natural density (g/cm³) | UCS (MPa) | Rock type Field no Sample depth (m) | Natural density (g/cm³) | UCS (MPa) |
|---------------------------|-------------------------|-----------|------------------------------------|-------------------------|-----------|
| LS 24.8 ~ 25.0            | 2.78                    | 48.1      | LS CXM1SDZK01#                     | 22.9 ~ 30.1             | 2.71      | 68.8      |
| LS 23.6 ~ 23.8            | 2.79                    | 45.3      | LS CXM1SDZK01#                     | 30.3 ~ 30.6             | 2.67      | 38.0      |
| LS 14.4 ~ 14.6            | 2.68                    | 64.5      | LS CXM1SDZK01#                     | 32.2 ~ 32.4             | 2.68      | 45.4      |
| LS 14.6 ~ 14.8            | 2.75                    | 32.4      | LS CXM1SDZK01#                     | 35.1 ~ 35.3             | 2.66      | 55.8      |
| LS 18.4 ~ 18.6            | 2.75                    | 38.0      | GR CXL2#DQZK2                      | 26.8 ~ 27.0             | 2.75      | 122.9     |
| LS 20.4 ~ 20.6            | 2.74                    | 42.1      | GR CXL2#DQZK2                      | 28.5 ~ 28.7             | 2.75      | 140.6     |
| LS 15.1 ~ 15.3            | 2.66                    | 44.7      | GR CXL2#DQZK2K4                    | 24.5 ~ 24.7             | 2.77      | 173.6     |
| LS 15.5 ~ 15.7            | 2.68                    | 48.2      | GR CXL2#DQZK2K4                    | 28.8 ~ 29.0             | 2.76      | 171.7     |
| LS 16.7 ~ 16.9            | 2.68                    | 52.0      | GR CXL2#DQZK2K4                    | 30.1 ~ 30.2             | 2.64      | 103.4     |
| LS 22.1 ~ 22.3            | 2.72                    | 56.8      | GR CXL2DQ2K5                       | 25.0 ~ 25.2             | 2.77      | 105.4     |
| LS 22.5 ~ 22.7            | 2.70                    | 60.5      | GR CXL2DQ2K5                       | 28.5 ~ 28.8             | 2.81      | 95.2      |

LS, limestone; GR, granite

The UCS increases gradually with the sample depth for the CTBDQZK3 borehole, while decreases with the sample depth for the CJS2DQZK3. For the CJS1DQZK1 borehole, both the natural density and the UCS of the argillaceous siltstone increase with the increase of the sample depth. For the CJS1DQZK2 borehole, the natural density broadly decreases with the sample depth. Except for the sample depth of 8.0 ~ 8.3 m, the UCS gradually increases with the sample depth. In general, the natural density of the argillaceous siltstone is smaller than that of the quartz sandstone, and the UCS of the former is much smaller than that of the latter. Therefore, compared with other rock types, the argillaceous siltstone has the smallest UCS, which indicates that it is most likely to be damaged for the excavated slopes. Also, the cohesion of the argillaceous siltstone is determined to be 3.25 MPa, and the internal friction angle is 53.7° based on the direct shear tests. Under excavation condition, the shear strength of the argillaceous siltstone is relatively small,
which is conducive to the sliding of rock strata. Therefore, in the excavation area of the slope, special attention should be paid to the excavation of the rock strata, and the supporting measures should be taken when necessary.

Deformation characteristics and professional monitoring

Field survey

The deformation characteristics of the exit slope of the tunnel before the excavation are presented in Fig. 5. The exit slope of the tunnel is steep, and its gradient is 65°. The thickness of the overburdened layer in this slope is approximately 3~5 m. Therefore, the exit slope at the interface of the overburdened layer and bedrock has the possibility of landslide under the influence of rainfall and artificial construction.

According to the field survey, the DRB is located near the exit of the right hole of Qingshuigou tunnel, as shown in Fig. 5. The elevation of the DRB is 1060~1140 m. The total volume is approximately 26,000 m³ with the height of 60~80 m, the width of approximately 25 m, and the thickness of 10~20 m. It can be seen from the geological longitudinal section of the exit slope of Qingshuigou tunnel that the DRB is very steep, upright, and partially overhanging. There are three groups of unloading cracks in the DRB, as shown in Fig. 5a and d. The sizes of these cracks are 2~25 cm width and 20~40 cm length, 5~30 cm width and 30~40 cm length, and 3~15 cm width and 20~30 cm length, respectively. The first unloading crack has 45° of the strike and 60° of the dip angle. The second unloading crack has 100° of the strike and 75° of the dip angle. Also, the third unloading crack has 355° of the strike and 80° of the dip angle.

The bedrock of the DRB is argillaceous siltstone, which belongs to soft rock. The strongly weathered rock mass has been obviously unloaded and loose, and the rock mass is relatively broken. The unloading cracks and small faults outside the gently inclined slope are developed, which correspond to the outside edge of the DRB in space. From the medium to slightly weathered zone, the rock mass is relatively complete and there are some fractures developed. In the upper loading area of the DRB, the rock mass has certain compression deformation under the load. The failure mode of the DRB is toppling and collapse failure, which is in an unstable state. The DRB with the main collapse direction of 0° to the gully or the exit direction of 270° to the right hole of the tunnel is in an unstable state. The DRB may be occurred to destroy under the induction of load, blasting vibration, and earthquake. If the DRB occurs to collapse, it has a great influence on the structures of rock mass at the exit of the tunnel under special conditions. Therefore, it is suggested to carry out earthwork clearing.

A landslide body is developed about 10 m away from the upper right side of the exit slope, which is a slump slope, as shown in Fig. 5b. The landslide body has a length of 30 m, a height of approximately 15~20 m, and a thickness of approximately 3~5 m. The volume of the landslide body is about 1000 m³. The exposed strata are light gray and dark gray thin to medium thick sandstone of the Niaoge Formation of the upper Triassic. The thickness of the strongly weathered layer is about 8~15 m. According to the field survey, the landslide occurred due to the influence of artificial excavation in road construction. The head scarp of the landslide body is excavated to form a steep slope with the height of approximately 20 m and the terrain slope exceeds 70°, and the strongly weathered argillaceous sandstone collapses under the condition of fracture cutting. At present, the landslide body is in an unstable state. However, the main sliding direction is on the right ditch of the exit slope and far away from the exit slope, which has little impact on constructing the exit slope of the Qingshuigou tunnel.

An overburdened body is developed about 80 m away from the right side of the exit slope, as shown in Fig. 5c. The overburdened body is mainly composed of clay and broken stone. The overburdened body has a length of 200 m, a

| Rock type | Sample depth (m) | Natural density (g/cm³) | UCS (MPa) | Rock type | Sample depth (m) | Natural density (g/cm³) | UCS (MPa) |
|-----------|------------------|------------------------|-----------|-----------|------------------|------------------------|-----------|
| QS        | 24.8~25.0        | 2.78                   | 58.1      | AS        | CJS1DQZK1#       | 24.8~25.0              | 2.65      |
| QS        | 23.6~23.8        | 2.79                   | 61.6      | AS        | CJS1DQZK1#       | 23.6~23.8              | 2.71      |
| QS        | 8.0~8.3          | 2.68                   | 70.8      | AS        | CJS1DQZK2#       | 8.0~8.3                | 2.66      |
| QS        | 12.2~12.4        | 2.67                   | 78.0      | AS        | CJS1DQZK2#       | 9.2~9.5                | 2.67      |
| QS        | 14.1~14.3        | 2.68                   | 76.5      | AS        | CJS1DQZK2#       | 12.4~12.7              | 2.63      |
| QS        | 16.2~16.4        | 2.66                   | 76.4      | AS        | CJS1DQZK2#       | 14.6~14.8              | 2.65      |
| QS        | 29.6~29.8        | 2.74                   | 71.8      | AS        | CJS1DQZK2#       | 24.8~25.0              | 2.62      |

AS, argillaceous siltstone; QS, quartz sandstone
weight of approximately 100 m and a thickness of approximately $6 \sim 15$ m. According to the field survey, the overburdened body is in overall stable state. There is a possibility of a landslide because the head scarp of the exit slope is steep, which has a little impact on the construction of the tunnel. However, the instability in the overburdened body is easy to cause landslide and debris flow under special conditions, which has a certain impact on the pier of Yashadi No.1 Bridge.

Geophysical analysis

According to the relevant specification (that is highway engineering geophysical exploration specification), geophysical exploration in this area was carried out. Duk-2a type high-density electrical method instrument is used for the high-density electrical detection. The Wenner device is used to collect $30 \sim 40$ layers, and the deepest detection depth is $40 \sim 100$ m. In the process of the field observation, to ensure the accuracy of field data collection, the monitoring of change trend of data at any time should be paid attention to, and the repeated observation of distortion data points and the investigation of various abnormal phenomena should be strengthened. During the continuous survey of high-density electrical profile, the repeated observation data accounts for about 25% of the total data, and the total mean square relative error is $\pm 1.65\%$, which meets the requirement of the specification.

The high-density electrical data is preprocessed before processing, and then the data is converted into

![Fig. 5](image-url) Slop structure outside and near the boundary of the slope source area. a The DRB. b The landslide body at the back scarp of the slope. c The overburdened body at the right boundary of the slope. d The unloading cracks in the middle of the slope.
two-dimensional inversion and topographic correction. Finally, surfer mapping is used for data interpretation. The two-dimensional inversion program is a computer inversion calculation program based on the smooth constrained least square method. It uses a new quasi-Newton optimization nonlinear least square algorithm, which makes the calculation speed several times faster than the conventional least square method in the case of a large number of data. This interpretation mainly refers to the apparent resistivity value profile, and uses the inversion results for comparison and reference. Apparent resistivity is a parameter that characterizes different conductive properties of substances. The main factors affecting the apparent resistivity include mineral composition, structure, and porosity.

To observe and study the distribution law of underground stable current field established manually, the high-density electrical method is used in the process, which is of great help to distinct the type of rocks, the water content, and the development of the cracks. In this study, a transverse profile of unstable slope at the exit section of Qingshuigou tunnel based on the high-density resistivity method is acquired by measuring the survey line along with the direction of the tunnel, as shown in Fig. 6. The starting point of the transverse profile is LK39+640 m, so the distance of the starting point on the profile is set as 0. The end point of the transverse profile is in LK39+980 m, so the distance of the end point on the plane is set as 360 m. The survey line is inclined to the axis direction of the Qingshuigou tunnel. According to the geophysical exploration results, the exposed strata is mainly grayish yellow, grayish green thin to medium thick siltstone and quartz sandstone in the Niaoge Formation of the upper Triassic. In Fig. 6, the area with different colors represents different apparent resistivity. The apparent resistivity near surface is about 140 ~ 240 Ω·M, which can reveal the thickness of overburdened layer about 0.5 ~ 3 m. At a distance of 220 ~ 360 m, the surface presents a mass of high resistivity body, which is explained by the backfill gravel of road construction. At a distance of 100 ~ 240 m and an elevation of 1170 ~ 1240 m, it presents a banded low resistivity body with apparent resistivity of approximately 40 ~ 140 Ω·M, which is interpreted as broken rock mass and speculated as differential weathering zone. At a distance of 25 ~ 60 m, an elevation of 1240 ~ 1260 m; a distance of 260 ~ 265 m, an elevation of 1180 ~ 1185 m; and a distance of 270 ~ 283 m, an elevation of 1155 ~ 1170 m, it is a low resistivity mass with apparent resistivity of about 40 ~ 140 Ω·M, which is interpreted as rock mass fragmentation.

In addition, longitudinal profile at the exit section of Qingshuigou tunnel based on the high-density resistivity method is acquired by measuring the survey line perpendicular to the axis direction of the tunnel, as shown in Fig. 7. The location of the longitudinal profile is LK39+991 m. According to the geophysical exploration results, the exposed strata is mainly limestone. The surface topography near the survey line is high in the middle and low at both ends. The apparent resistivity of the overburdened near the surface is about 10 ~ 100 Ω·m and the thickness is about 1 ~ 7 m. At the horizontal distance of 100 m, 150 m, and 200 m, there is low resistivity anomaly zone, which is supposed to be the dissolution fracture zone.

Fig. 6 Transverse profile of unstable slope at the exit section of Qingshuigou tunnel based on the high-density resistivity method
According to the statistics and analysis, the variation range of the apparent resistivity value of the overburdened layer is not large, and the overburdened layer mostly has low resistivity. The variation range of the apparent resistivity value of the non-soluble rocks, such as sandstone and granite, varies greatly. The apparent resistivity value of strongly weathered rock mass near the surface or the fractured zone of rock mass is generally less than 500 Ω·m due to the high water content. The apparent resistivity value of relatively complete rock mass is higher, mostly more than 2000 Ω·m. The apparent resistivity value of compact rock mass in some sections is more than 5000 Ω·m. Nevertheless, the compact rock mass may also show the characteristics of high resistivity due to the case that dissolution cavity is not completely filled with water and mud. Moreover, mineralization and carbonaceous phenomena exist in some areas, and the apparent resistivity value is generally low, which is easy to cause interference anomalies. On the whole, combined with the geological data and statistics of the physical parameters of rock mass in this area, the apparent resistivity of different media is determined, as shown in Table 3.

### Table 3 The apparent resistivity of different media in this area

| No | Type              | Apparent resistivity (Ω·m) |
|----|-------------------|-----------------------------|
| 1  | Overburdened layer| 20~600                      |
| 2  | Sandstone         | 400~3000                    |
| 3  | Granite           | 80~6000                     |
| 4  | Limestone         | 100~7000                    |
| 5  | Dolomite          | 80~6500                     |
| 6  | Groundwater       | <100                        |

**Monitoring analysis**

During the field survey, several methods, such as on-site monitoring and geophysical prospecting, have been applied to the exit slope. Since 2019, on-site monitoring of the exit slope has been carried out many times. Slope monitoring is mainly composed of surface deformation and deep deformation. Among them, the surface deformation monitoring means are macroscopic geological phenomenon monitoring and total station measurement monitoring. The deep deformation monitoring means are mainly borehole inclination monitoring. The monitoring points are arranged into four monitoring sections according to the left and right hole of the tunnel and its center line. The position of each monitoring point is shown in Fig. 8. The first monitoring section is located near the left hole of the tunnel. The second monitoring section is located near the left side of the center line of the tunnel, while the third monitoring section is located near the right side of the center line of the tunnel. The fourth monitoring section is located near the right hole of the tunnel. By monitoring the surface deformation, the deep deformation and the macro geological phenomena of the slope in an all-round and multi-means way, the deformation and failure situation of the slope can be mastered in real time, and the treatment effect can be evaluated, which can provide reliable monitoring data and decision-making basis for the risk assessment and disaster warning of the slope.
Surface displacement

To obtain surface displacement vectors presented in Fig. 8 and cumulative surface displacement curves based on the monitoring results in Fig. 9, the monitoring points CD14, CD15, CD16, CD20, and CD22 are classified into the first monitoring section; the monitoring points CD1, CD3, CD5, CD7, CD12, CD13, and CD17 are classified into the second monitoring section; the monitoring points CD2, CD4, CD6, CD18, CD21, and CD24 are classified into the third monitoring section; and the monitoring points CD8, CD9, CD10, CD11, and CD19 are classified into the fourth monitoring section. The distribution of the monitoring points is relatively concentrated. The monitoring period is from November 11, 2019, to May 1, 2020. The data collection time is basically once a day in November, 2019. In the periods from December 1, 2019, to January 1, 2020 and from March 8, 2020, to May 1, 2020, data are collected every 2–4 days. Noteworthy, due to the impact of 2019-nCoV, no monitoring data are collected during the period from January 1, 2020 to March 8, 2020. Noteworthy, the default value of the data is set in March 5, 2022, when drawing the surface displacement curves. Therefore, after 2 months, the surface displacement was collected again, and it rises sharply. The reason is that the frequency of data acquisition from the monitoring is different. The results show that the cumulative surface displacements for all monitoring points range from 120 to 200 mm during the monitoring period. Concretely, the cumulative surface displacements of CD20 and CD22 are larger than those of other monitoring points for the first monitoring section. The CD20 and CD22 are located at the head scarp and back scarp of the first monitoring section, respectively. For the second monitoring section, the cumulative surface displacement of CD17 is larger than that of other monitoring points, and the displacement of CD12 is minimal. The CD12 and CD17 are located at the head scarp and back scarp of the second monitoring section, respectively. For the third monitoring section, the cumulative surface displacement of CD18 is larger than that of other monitoring points, and the displacement of CD24 is minimal. The CD24 and CD18 are located at the head scarp and back scarp of the third monitoring section, respectively. For the fourth monitoring section, the cumulative surface displacement of CD11 is the smallest among these monitoring points. Therefore, the deformation at the head scarp of the exit slope is relatively small, and these monitoring points are located in relatively stable areas. This is because some relevant reinforcement measures have been taken to prevent the deformation of the slope. These monitoring points with relatively large deformation are located outside the reinforced slope area. Thus, the deformation of these points is obvious. These regions exhibit slow creep deformation. The cumulative surface displacement of the exit slope from the head scarp to the back scarp increases from 115 to 125 mm to 165 to 180 mm, presenting pushing deformation characteristics. Although the surface displacement at the back scarp of the exit slope is larger than that at the head scarp of the exit slope, the slope is still in a stable state, and the deformation of the slope tends to be more and more stable. Combined with the field survey, the monitoring results are in well agreement with the deformation phenomenon, such as tensile cracks and unloading cracks.
Deep displacement

To obtain deep displacement vectors presented in Fig. 8 and cumulative deep displacement curves based on the monitoring results in Fig. 10, the monitoring points of determining the deep displacement are the same as those of determining the surface displacement. The time and frequency of the deep displacement monitoring data collection of the exit slope are consistent with those of the surface displacement data collection. As shown in Fig. 10, the deep deformation of the slope can be divided into two periods. The first period is from November 11, 2019, to March 8, 2020. During this period, the cumulative deep displacement gradually increases with the time. The exit slope exhibits creep deformation. The reason is that the excavation of the slope has been completed or will be completed soon, so some unloading cracks appear during the period. The second period is from March 8, 2020 to May 1, 2020. During the period, the exit slope enters the stable deformation stage. The reason is that after the first period, local excavation of the slope and the reinforcement are simultaneously carried out. Similarly, the deep deformation at the head scarp of the slope, such as CD15, CD16, and CD22 at the first monitoring section, CD12 at the second monitoring section, CD24 at the third monitoring section, and CD11 at the fourth monitoring section, is smaller than that at the back scarp of the slope. The maximum cumulative deep displacement of CD22 is 119.2 mm while the maximum cumulative deep displacement of CD20 is 196.8 mm at the first monitoring section. The maximum cumulative deep displacement of CD12 is 59.9 mm while the maximum cumulative deep displacement of CD7 is 217.1 mm at the second monitoring section. The maximum cumulative deep displacement of CD24 is 69.6 mm while the maximum cumulative deep displacement of CD6 is 208.5 mm at the third monitoring section. The maximum cumulative deep displacement of CD11 is
35.3 mm while the maximum cumulative deep displacement of CD19 is 133.8 mm at the fourth monitoring section. Obviously, the deep displacement at the back scarp is 2–4 times of that at the head scarp of the slope. On the whole, the deformation of the slope tends to be more and more stable.

Discussion

Deformation mechanism

Hydrogeologic conditions

There exists widely surface water and groundwater in the study area. Surface water mainly comes from rivers, lakes, and atmospheric precipitation, etc. The annual average precipitation of Gejiu City is 1068.7 mm. The abundant precipitation is not conducive of the stability of the slope. The horizontal distribution of annual precipitation in Gejiu City is not different, but the difference is obvious with different altitude. It is mainly manifested in that there is more rainfall in the mountainous area and less rainfall in flat area. The rainfall of Gejiu City in November is higher than that in January, February, and March, while it is almost the same as that in April, but lower than that in May. Therefore, it can be seen from the monitoring results that the increase rate of deformation in November is significantly higher than that in January, February, and March so that the rainfall can aggravate the deformation of the slope. The tunnel site is characterized by high mountains, steep slopes, and deep gully cutting. There is no spring point in the area. The atmospheric precipitation rapidly turns into a surface runoff, and rapidly enters the underground along the joints and fissures. Therefore, in addition to the rainstorm season, the surface runoff in the tunnel site is rare. Although Jiasha river is developed in the west side of the tunnel, with an elevation of 872 m, the surface runoff has a little impact on the tunnel with the lowest elevation of 1000 m.

Fig. 10 Monitoring results of the cumulative deep displacement. a Displacement near the left hole of the tunnel. b Displacement at the left side of the center line of the tunnel. c Displacement at the right side of the center line of the tunnel. d Displacement near the right hole of the tunnel.
The groundwater in the study area is controlled by lithology, structure, landform, and other factors, and the recharge of the groundwater is closely related to rainfall. According to the occurrence conditions of groundwater, it can be divided into two types: loose layer pore water and bedrock fissure water. According to the field survey, the pore water of the Quaternary loose layer mainly occurs in the loose overburdened layer of the slope and ditch bottom. The thickness of the clay layer is different in different zones, and its permeability is medium. After, the gullies receive the atmospheric precipitation. Part of the atmospheric precipitation is discharged to the downstream low-lying area along the gully, and part of the atmospheric precipitation is discharged downward in the form of surface flow and discharged to the deep through the bedrock fissures, so the water content in the clay layer is less and there is no water in the dry season. Therefore, the content of loose layer pore water is small and varies greatly with seasons, so it has no impact on the construction and operation of the tunnel. The groundwater in the study area is mainly hosted in weathering fissures and structural fissures of rock mass, which belongs to bedrock fissure water. Affected by the regional structure, the structural fissures and weathering fissures of the rock mass in the area are relatively developed. The precipitation flows along the surface of the slope, and then moves to the deep along the fissures after infiltration. On the whole, the tunnel is located in the mountainous area, the terrain is high and steep, and the groundwater storage conditions are poor. When the atmospheric precipitation is replenished, it cannot be stored and discharged rapidly, so the groundwater level is buried deep. According to the above analysis, most of the groundwater during the excavation of the tunnel is drip and linear drainage, and there will be no water intrush. As long as the effective drainage measures are taken, the groundwater has a little influence on the excavation of the tunnel.

To further reveal the influences of the water on the concrete structure in the study area, the tests of determining the chemical characteristics of the water are conducted. According to the analysis of water samples near the study site, it is shown that the types of the surface water quality in the tunnel area are mainly $\text{HCO}_3^– + \text{Na}^+ + \text{K}^+ \cdot \text{Ca}^{2+}$, with a PH value of 6.7 and total salinity of 218.6 mg/L. The hydrochemical characteristics in this study site are presented in Table 4.

According to the evaluation standard of highway engineering geological investigation specification (JTG C20-2011), the corrosivity of water and soil to concrete structure can be evaluated according to environment type and stratum permeability. The corrosivity evaluation of water and soil to reinforcement in reinforced concrete structure can also be carried out. The results reveal that the corrosivity of water and soil to the concrete structure is micro corrosivity according to the environmental type, and the corrosivity of water and soil to the concrete structure is evaluated as micro corrosivity. According to the corrosivity of water and soil to reinforcement in reinforced concrete structure, it is also evaluated as micro corrosivity.

### Natural weathering

Natural weathering of rock mass is mainly affected by lithology, structure, vegetation, climate, and water, and many other factors, so it is very important to analyze which factors play an important role in the weathering of rock mass in the exit slope. Due to the steep original terrain of the slope, the cracks of rock mass under the long-term natural unloading condition are developed, and the rock mass is relatively broken. Due to the existence of soft and hard interbedded rock mass, the weathering difference in the study area is obvious. The weathering difference further leads to the poor integrity of the rock mass, resulting in the development of the original adverse geological phenomena of the slope. In addition, due to the weight unloading of the exit slope, the unloading rebound effect of the slope excavation may cause tensile deformation of the surface rock mass and the displacement to the west or southwest under the joint action of self-weight unloading.

According to the XRD results and petrographic identification characteristics of the rocks in the tunnel site, it can be considered that the weathering of rock mass is relatively weak, and most of the weathering occurs in the brittle structural zone and small fault development zone (Wen et al. 2020b, c). The weathering of rock mass is mainly manifested in the dissolution and transformation of minerals in original

| Cation content (mmol/L) | Anion content (mmol/L) |
|------------------------|------------------------|
| Na$^+$ + K$^+$ | Ca$^{2+}$ | Mg$^{2+}$ | Cl$^–$ | SO$_4^{2–}$ | HCO$_3^–$ | CO$_3^{2–}$ | Water temperature ($°$) | Bicarbonate alkalinity (mmol/L) | Carbonate alkalinity (mmol/L) | Total alkalinity (mmol/L) |
| 1.518 | 1.342 | 0.242 | 0.0839 | 0.352 | 2.5696 | 0.0966 | 16 | 2.5696 | 0.0966 | 1.3814 |

218.6 | 6.7 | 1.584 | 0.242 | 0.0839 | 0.352 | 2.5696 | 0.0966 | 16 | 2.5696 | 0.0966 | 1.3814 |
broken rocks and altered rocks by various anions and cations in groundwater and the formation of secondary minerals.

Combined with the field survey, the weathering degree of rock mass in the exit slope has the following characteristics: in the horizontal direction, the weathering degree of rock mass from the surface of the slope to the interior of the slope decreases; in the vertical direction, the weathering degree of rock mass increases with the increase of the elevation. The weathering degree is greatly different due to the influence of many factors. The weathering of rock mass is obviously stronger on both sides of the fault, and the rust staining of rocks in this area is obvious. The weathering of rock mass near the fracture plane is relatively weak, which mainly occurs along the fracture plane.

**Excavation activities**

The exit of Qingshuigou tunnel is located in the exit slope. The tunnel has two exits: right hole and left hole. The left hole of Qingshuigou tunnel has been completed on December 2019, and the right hole of Qingshuigou tunnel has about 100 m left by January 15, 2020. At present, the right hole of Qingshuigou tunnel has been completed. According to the results monitoring and measurement data in the tunnel, there is no abnormal deformation on the left hole of the tunnel, and no deformation signs, such as cracking and spalling, are found. However, according to the results revealed by the excavation of the right hole, the surrounding rock is thin to medium thick-bedded sandstone or argillaceous siltstone or mudstone of the Niaoge Formation of the upper Triassic. It has the phenomenon of soft and hard interbedding. The rock mass is generally gently inclined to the west, and the occurrence of the rock stratum is 210°±18°. Due to the development of cracks, the rock mass is relatively broken. Water seepage or dripping can be seen in the rock mass, leading to vault collapse and other problems in the tunnel.

To maintain the stability of the exit slope due to the excavation of the tunnel, the exit slope has been excavated and supported, and the slope height is approximately 50 m. Owing to the continuous excavation activities, there exists an unloading phenomenon of rock mass. The unloading of rock mass refers to that when the free surface is formed due to the undercutting of a river valley or artificial excavation, the strain energy stored in the rock mass begins to release, and the unloading is produced towards the free surface, which leads to the rebound of rock mass. Afterwards, with the process of the unloading and rebound, new structural planes are formed in the rock mass within a certain depth of the slope, or along the existing structural plane, which eventually leads to the rock mass in a certain depth range of the slope to become relaxed and tensioned.

The essence of the unloading is that the original stress in the original rock decreases, which makes the structure of rock mass become relaxed. With the gradual unloading and stress reduction of the rock mass, new cracks will gradually form in this process. These cracks provide a channel for the weathering, further accelerating the weathering process of the rock mass and causing the further decline of the stress of the original rock. Therefore, the rock mass will be further damaged.

Due to the special geographic environment of the study area, the bedrock of the slope often forms broken loose rock mass under various actions (such as the weathering unloading and freeze–thaw), and the thickness and scale of these broken loose rock mass are mostly different. In the study area, the deeper the cataclastic rock mass is, the worse the stability of the slope is. On the right side of the excavation slope, the structure of the broken loose rock mass is mostly fragmentary loose, and the distribution range of the broken loose rock mass is relatively small. The fractured loose rock mass is widely distributed in the downstream of the right side of the slope. There are collapse and slide cliffs in some areas, which are mostly in cataclastic-loose structure. The massive rock mass is scattered in the distribution range, which plays a supporting role in the cataclastic-loose rock mass.

In the shallow part of the slope, the rock mass of the slope presents the characteristics of the cataclastic structure due to the combined action of primary columnar joints and unloading cracks. Under the action of a gravity field, the cataclastic rock mass is prone to collapse to the free surface.

According to the adit survey, the unloading cracks in the slope are mainly caused by the unloading and rebound of the rock mass on the bank slope under the premise of cutting down the river valley, and the dip angle of the unloading cracks is mostly steep. On the whole, the unloading of rock mass in the study area generally shows that the unloading phenomenon at high elevation is more obvious than that at low elevation, and the unloading at the shallow horizontal depth is more obvious than that at the deep level.

**Stabilization treatment**

Due to the existing deformation of the exit slope, it is very important to take some measures to prevent the exit slope from further deformation. At present, the exit slope of the Qingshuigou tunnel has been excavated and supported, and the height of the slope is about 50 m, as shown in Fig. 11. Firstly, the dangerous rock mass on the right side of the tunnel has been partially removed. Afterwards, the exit slope is divided into five levels of berm to take bolt-shotcreting support measures. In the process of the slope excavation, the unloading cracks develop obviously. According to the monitoring results mentioned above, the cracks in the slope are mainly longitudinal joints. Unloading by the impact of the excavation, the displacement of
the slope is mainly towards the gully on the left side of the left tunnel, and the displacement in the overhead direction of the slope is small. The closer to the slope, the greater the deformation. The 4–6 m strongly weathered siltstone below the finished surface of the excavated slope, and then the moderately weathered siltstone below, so the anchoring effect is fairly good. The prestressed anchor frame beam is used to reinforce the first- to third-grade front slope above the tunnel entrance. The anchorage section of the reinforcement material should be embedded into the moderately weathered rock mass to strengthen the connection between the slope surface and the deep moderately weathered rock mass. Meanwhile, the transverse rock mass is strengthened by a frame beam to control the development of longitudinal cracks. According to the geological longitudinal section, the bolt’s length is determined to be 12 m, and the design tensioning tonnage is 120 kN. The typical support section is shown in Fig. 11, and the support area is approximately 1700 m². The properties of the support measurements are presented in Table 5.

As for the treatment of the right side of the right hole of the tunnel, the dangerous rock mass has been partially removed. After cleaning, the slope surface is exposed as strongly weathered siltstone, argillaceous siltstone, and silty shale. The slope is a reverse slope with alternating soft and hard rocks, obvious differential weathering, and relatively developed cracks. In order to ensure the safety of the tunnel operation, it is necessary to reinforce the slope. Firstly, the relatively broken rock mass in the slope surface is removed, then the prestressed anchor cable frame beam is used to reinforce the slope surface with high development degree of local unloading cracks. The reinforced slope area is 400 m², presented in the green zone in Fig. 12a. Due to the large undulation of the slope surface, the frame beam shall be erected close to the existing slope surface. The length of the anchor cable is 20 m, and the embedded angle is 20°. The anchorage section of reinforcement material should be embedded into the moderately weathered rock mass. According to the geological longitudinal section, the bolt’s length is determined to be 10 m, and the design tensioning

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**Table 5** The properties of the support measurement used in the slope

| Type                     | Location                                      | Length/m | Embedded dip angle/° |
|--------------------------|-----------------------------------------------|----------|----------------------|
| Prestressed anchor frame beam | The front slope on the right side of the tunnel: the slope ratio 1:1 | 20        | 20                   |
| Prestressed anchor frame beam | The front slope on the right side of the tunnel: the slope ratio 1:0.5 | 15        | 20                   |
| Prestressed anchor cable frame beam | Above platform of second front slope           | 20        | 20                   |
| Prestressed anchor cable frame beam | Above sidewalk                               | 20        | 20                   |
| Prestressed anchor frame beam                   | Above sidewalk                               | 12        | 20                   |
tonnage is 300 kN. After implementing the anchor cable frame beam, the whole slope (red zone in Fig. 12a) on the right side of the tunnel is closed by hanging net and shotcrete to prevent the slope from further weathering during the operation period of the tunnel. The length of the bolt is 3 m and the thickness of shotcrete is 10 cm. The slope area after reinforcement is about 3000 m².

As for the treatment of the left side of the left hole of the tunnel, the slope is divided into two levels of a berm to take bolt-shotcreting support measures. According to the monitoring results mentioned above, the cracks in the slope are mainly longitudinal joints, and the displacement in the overhead direction of the slope is also small. Similarly, due to the 4–6 m strongly weathered siltstone below the finished surface of the excavated slope, and then the moderately weathered siltstone below, the anchoring effect is fairly good. Considering the relative position of the slope and the tunnel, the prestressed anchor frame beam is used to reinforce the first front slope above the tunnel entrance. According to the geological longitudinal section, the bolt’s length is determined to be 12 m, and the design tensioning tonnage is 120 kN, as shown in Fig. 12b. Afterwards, the prestressed anchor cable frame beam is used to reinforce the second front slope above the tunnel entrance. The length of the anchor cable is determined to be 20 m, and the design tensioning tonnage is 300 kN. The typical support section is shown in Fig. 12b, and the support area is approximately 700 m². During the implementation, the relationship between the support and the tunnel section should be further checked to avoid the anchor cable stretching into the tunnel.

Furthermore, the unloading cracks in the slope between the left and right holes of the tunnel are relatively developed, and the slope surface is relatively broken. In order to ensure the later operation safety and road uniformity, some measures should be taken. As for the treatment of the slope between left and right holes of the tunnel, the slope should be cleaned to the design elevation of subgrade, and the disturbance to the whole slope should be reduced as much as possible during the cleaning construction. The excavation volume in this zone is about 450 m³, as shown in Fig. 12c.

As for the treatment of the slope below the tunnel exit, the broken slope surface is cleaned up, and the prestressed anchor frame beam is used to reinforce the steep slope below the elevation of the portal subgrade, as shown in Fig. 12d and e. Due to the large undulation of the slope surface, the frame beam shall be erected close to the existing slope surface. The bolt’s length is determined to be 12 m, and the design tensioning tonnage is 120 kN. After implementing the anchor frame beam, the whole slope surface is closed by hanging net and shotcrete. Herein, the bolt’s length is also 3 m and the thickness of shotcrete is also 10 cm. The slope area after reinforcement is about 3000 m². The slope area after the treatment of the anchor frame beam is about 2000 m², and that of shotcreting and anchoring is about 3000 m².

To further explore and quantitatively evaluate the stability of the exit slope under excavation, limit equilibrium method is utilized to analyze the changes in the global stability status. The cross section of the front slope on the left side of the left hole of the tunnel (indicated in Fig. 11b) is chosen as the typical section for calculation. The cross section is subjected to excavation disturbances. In this study, by simplifying the geologic model, the stability analysis of the section of the exit slope before and after the reinforcement of the slope under the excavation construction is carried out by using the software based on the limit equilibrium method.

Based on the calculating results, the stability safety factor of the slope is smaller and smaller with the sliding plane moving from top to bottom before the reinforcement of the slope. After the reinforcement, the stability of the slope is obviously enhanced. Combined with the analysis of the geological characteristics of the slope, the slope at this section
is a gently inclined outer layered slope, and the dip angle at the toe of the slope the soft surface of the overburdened is similar. It is preliminarily determined that if the slope is not supported under the excavation construction, the slope may slide along the underlying weak surface.

Conclusions

The comprehensive research outlined in the study is aimed at understanding the deformation and failure mechanisms of the exit slope that excavated in 2019 and 2020. These fields are discussed through the comprehensive research including details field survey, monitoring analysis, deformation mechanism, and stabilization treatment. Based on the field survey and analysis, the conclusions are as follows:

(1) The results of the field survey reveal that the exit slope at the interface of the overburdened layer and bedrock has the possibility of landslide through the joint force of rainfall and artificial construction. The failure mode of the DRB is toppling and collapse failure, which is in an unstable state. The DRB may be occurred to destroy under the induction of load, blasting vibration, and earthquake, which has a great impact on the structures under the exit section of the tunnel. Therefore, it is suggested to carry out earthwork clearing.

(2) Based on the monitoring analysis, the surface displacement at the back scarp of the exit slope is larger than that at the head scarp of the exit slope; the deep displacement at the back scarp is 2–4 times of that at the head scarp of the slope, but the deformation of the slope still tends to be more and more stable. In addition, the variation range of the apparent resistivity value of the overburdened layer is not large, and the overburdened layer mostly has low resistivity. The compact rock mass may also show the characteristics of high resistivity due to the case that dissolution cavity is not completely filled with water and mud.

(3) Multiple factors are responsible for the gradual deformation of the exit slope, which can be summarized as hydrogeologic conditions, natural weathering, and excavation activities. Most of the groundwater during the excavation of the tunnel is drip and linear drainage; the groundwater has a little influence on the excavation of the tunnel as long as the effective drainage measures are taken. For the natural weathering, the weathering degree of rock mass from the surface of the slope to the interior of the slope decreases in the horizontal direction; in the vertical direction, the weathering degree of rock mass increases with the increase of the elevation. For the excavation activities, the unloading phenomenon at high elevation is more obvious than that at low elevation, and the unloading at the shallow horizontal depth is more obvious than that at the deep level.

(4) Some measures are taken to prevent the exit slope from further deformation. Firstly, the relatively broken rock mass in the slope surface is removed, then the prestressed anchor cable frame beam is used to reinforce the slope surface with high development degree of local unloading cracks.

Author contribution  Tao Wen contributed to the conception of the study; Zheng Hu provided the data; Huiming Tang helped perform the analysis with constructive discussions.

Funding  The work was funded by the National Natural Science Foundation of China (No. 42002268); Cooperative Innovation Center of Unconventional Oil and Gas (Ministry of Education & Hubei Province) (UOG2020-11); Open Foundation of Engineering Research Center of Rock-Soil Drilling & Excavation and Protection, Ministry of Education; and The 14th college students’ Innovation and Entrepreneurship Training Program of Yangtze University.

Declarations

Conflict of interest  The authors declare no competing interests.

References

Chigira M, Wu XY, Inokuchi T, Wang GH (2010) Landslides induced by the 2008 Wenchuan earthquake, Sichuan, China. Geomorphology 118(3–4):225–238
Cogan J, Gratchev I, Wang GH (2018) Rainfall-induced shallow landslides caused by ex-tropical cyclone Debbie, 31st March 2017. Landslides 15(6):1215–1221
Du SG, Lin H, Yong R, Liu GJ (2022) Characterization of joint roughness heterogeneity and its application in representative sample investigations. Rock Mech Rock Eng 1–25
Gorum T, Fan XM, van Westen CJ, Huang RQ, Xu Q, Tang C, Wang GH (2011) Distribution pattern of earthquake-induced landslides triggered by the 12 May 2008 Wenchuan earthquake. Geomorphology (amsterdam, Netherlands) 133(3–4):152–167
Gu DM, Huang D, Yang WD, Zhu JL, Fu GY (2017) Understanding the triggering mechanism and possible kinematic evolution of a reactivated landslide in the Three Gorges Reservoir. Landslides 14(6):2073–2087
Lee S, Pradhan B (2007) Landslide hazard mapping at Selangor, Malaysia using frequency ratio and logistic regression models. Landslides 4(1):33–41
Li ZQ, Xue YG, Li SC, Zhang LW, Wang D, Li B, Zhang W, Ning K, Zhu JY (2017) Deformation features and failure mechanism of steep rock slope under the mining activities and rainfall. J Mt Sci-Engl 14(1):31–45
Li CD, Fu ZY, Wang Y, Tang HM, Yan JF, Gong WP, Yao WM, Criss RE (2019) Susceptibility of reservoir-induced landslides and strategies for increasing the slope stability in the Three Gorges Reservoir Area: Zigui Basin as an example. Eng Geol 261:105279
Li Q, Wang YM, Zhang KB, Yu H, Tao ZY (2020) Field investigation and numerical study of a siltslope instability induced by excavation and rainfall. Landslides 17(6):1485–1499

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Lin F, Wu LZ, Huang RQ, Zhang H (2018) Formation and characteristics of the Xiaoba landslide in Fuquan, Guizhou, China. Landslides 15(4):669–681

Ma J, Su A, Zhang J, Wen T, Wang Y (2019) Reliability analysis for a large and complex landslide in the Three Gorges Reservoir Area (China) based on incomplete information. Geomatics Nat Hazards Risk 10(1):181–196

Ma J, Wang Y, Niu X, Jiang S, Liu Z (2022) A comparative study of mutual information-based input variable selection strategies for the displacement prediction of seepage-driven landslides using optimized support vector regression. Stoch Environ Res Risk Assess 1–21

Regmi AD, Yoshida K, Cui P, Hatano N (2017) Development of Taprang landslide, West Nepal. Landslides 14(3):929–946

Shi GC, Wang YF, Wang YK, Tao ZG, Wan LP, Xi LY (2020) Numerical analysis and deformation mechanism study on an excavated high-steep slope of a hydropower station. Adv Civ Eng 2020:1–17

Stähli M, Sättele M, Huggel C, McArdell BW, Lehmann P, Van Herwijnen A, Berne A, Schleiss M, Ferrari A, Kos A, Or D, Springman SM (2015) Monitoring and prediction in early warning systems for rapid mass movements. Nat Hazard Earth Sys 15(4):905–917

Tang HM, Zou ZX, Xiong CR, Wu YP, Hu XL, Wang LQ, Li CD (2015) An evolution model of large consequent bedding rockslides, with particular reference to the Jiweishan rockslide in Southwest China. Eng Geol 186:17–27

Tang HM, Wasowski J, Juang CH (2019) Geohazards in the three Gorges Reservoir area, China-lessons learned from decades of research. Eng Geol 261:105267

Wang Y, Tang H, Huang J, Wen T, Ma J, Zhang J (2022) A comparative study of different machine learning methods for reservoir landslide displacement prediction. Eng Geol 298:106544

Wang Y, Tang H, Wen T, Ma J, Zou Z, Xiong C (2019) Point and interval predictions for Tanjiahe landslide displacement in the Three Gorges Reservoir Area, China. Geofluids

Wen T, Hu Z, Wang Y, Zhang Z (2022a) Variation law of air temperature of a high-geotemperature tunnel during the construction. Lithosphere 2022(Special 7)

Wen T, Hu Z, Wang Y, Zhang Z, Sun J (2022b) Monitoring and Analysis of Geotemperature during the Tunnel Construction. Energies 15(3):736

Wen T, Tang H, Huang L, Wang Y, Ma J (2020a) Energy evolution: a new perspective on the failure mechanism of purplish-red mudstones from the Three Gorges Reservoir area, China. Eng Geol 264:105350

Wen T, Tang H, Huang L, Hamza A, Wang Y (2021) An empirical relation for parameter mi in the Hoek-Brown criterion of anisotropic intact rocks with consideration of the minor principal stress and stress-to-weak-plane angle. Acta Geotech 16(2):551–567

Wen T, Tang H, Wang Y, Ma J (2020b) Evaluation of methods for determining rock brittleness under compression. J Nat Gas Sci Eng 78:103321

Wen T, Tang H, Wang Y (2020c) Britteness evaluation based on the energy evolution throughout the failure process of rocks. J Petrol Sci Eng 194:107361

Wen T, Tang H, Wang Y, Ma J, Fan Z (2019) Mechanical characteristics and energy evolution laws for red bed rock of Badong Formation under different stress paths. Adv Civ Eng

Yu HB, Li CD, Zhou JQ, Chen WQ, Long J, Wang XT, Peng T (2020) Recent rainfall- and excavation-induced bedding rockslide occurring on 22 October 2018 along the Jian-En expressway, Hubei, China. Landslides 17(11):2619–2629