

Failure of a rock slope 16.5 years after excavation in repeated strata of sandstone and mudstone

Jun-Jie Wang¹ · Tian-Long Zhao² · He-Jun Chai³ · Sheng-Chuan Tang⁴

Received: 16 May 2016 / Accepted: 14 November 2016 / Published online: 19 November 2016
© Springer-Verlag Berlin Heidelberg 2016

Abstract The present study focuses on failure of a rock slope excavated in the autumn of 1997 and located in the repeated strata of sandstone and mudstone in Chongqing of China. The rock slope failed suddenly on April 28, 2014. The elapsed time from excavation to failure is about 16.5 years. The failure was sudden and unexpected seemingly, but it was mainly resulted from the initial excavation and the subsequent weathering in the long term. The initial excavation in 1997 as a reason why the rock cut slope failed was undoubted, because the excavation broke original ground stress balance, resulted in inducement or expansion of cracks and induced deformation and looseness of local rock masses. The weathering process, especially physical weathering process in the long term

(16.5 years after excavation), is the most important reason of the failure, because it resulted in disintegration of material on slope surface and induced reduction of strength in the interior of the slope. Water, as a key factor speeding up the weathering process, weakened mechanical properties of rocks and discontinuities and urged formation of argillation intermediate layer. Based on the analyses of failure modes and corresponding factors of safety, an integrated stabilization system on the rock slope was put in place.

Keywords Rock slope · Failure · Excavation · Weathering · Stabilization

This article is part of a Topical Collection in Environmental Earth Sciences on “Environment and Health in China II”, guest edited by Tian-Xiang Yue, Cui Chen, Bing Xu and Olaf Kolditz.

✉ Jun-Jie Wang
wangjunjiehu@163.com

- ¹ National Engineering Research Center for Inland Waterway Regulation, Chongqing Jiaotong University, Chongqing 400074, People’s Republic of China
- ² Key Laboratory for Hydraulic and Waterway Engineering of Ministry of Education, Chongqing Jiaotong University, Chongqing 400074, People’s Republic of China
- ³ National Engineering Research Center for Highways in Mountain Area, China Merchants Chongqing Communications Technology Research and Design Institute Co. Ltd., Chongqing 400067, People’s Republic of China
- ⁴ National Engineering Laboratory for Road Engineering and Disaster Prevention and Reduction Technology in Mountainous Areas, China Merchants Chongqing Communications Technology Research and Design Institute Co. Ltd., Chongqing 400067, People’s Republic of China

Introduction

The instabilities of slopes or landslides are usually extraordinary serious hazards for human activities because of causing property damages, maintenance costs, economic losses and injuries or fatalities (Hoek and Bray 1981; Pantelidis 2009). The stability and failure mode of rock slopes are often affected by many factors such as geological structures (such as folds, faults and discontinuities) (Chai et al. 2009; Babiker et al. 2014; Stead and Wolter 2015), seismicity (Brain et al. 2014), physical and chemical weathering (Admassu et al. 2012; Mišcevic and Vlastelica 2014) and excavation (Wang et al. 2008). The structural features control rock mass behavior and contribute to either the stabilization or destabilization of rock slopes (Glastonbury and Fell 2000; Stead et al. 2006). Earthquake may trigger slope failures, and the cumulative effect of regional seismicity may damage and weaken slopes (Moore et al. 2011; Wang et al. 2013b). Both physical and chemical weathering and alteration are caused by temperature

changes and fluid circulations within the fractures and pores (Hall and André 2001; Bouissou et al. 2012). The weathering process may result in the disintegration of material on slope surface and results in the reduction of shearing strength in the interior of the slope (Duperret et al. 2005; Chigira and Yokoyama 2005). The method of excavation is also a factor influencing the stability of rock slope. It is involved, as a parameter, in many rock mass classification systems (Liu and Chen 2007) such as the slope mass rating (SMR) (Romana 1985), Chinese slope mass rating (CSMR) (Chen 1995) and slope stability probability classification (SSPC) (Hack et al. 2003). The time since excavation may also be a factor related to the safety of rock slope (Nicholson 2004), but not involved in these rock mass classification systems (Pantelidis 2009).

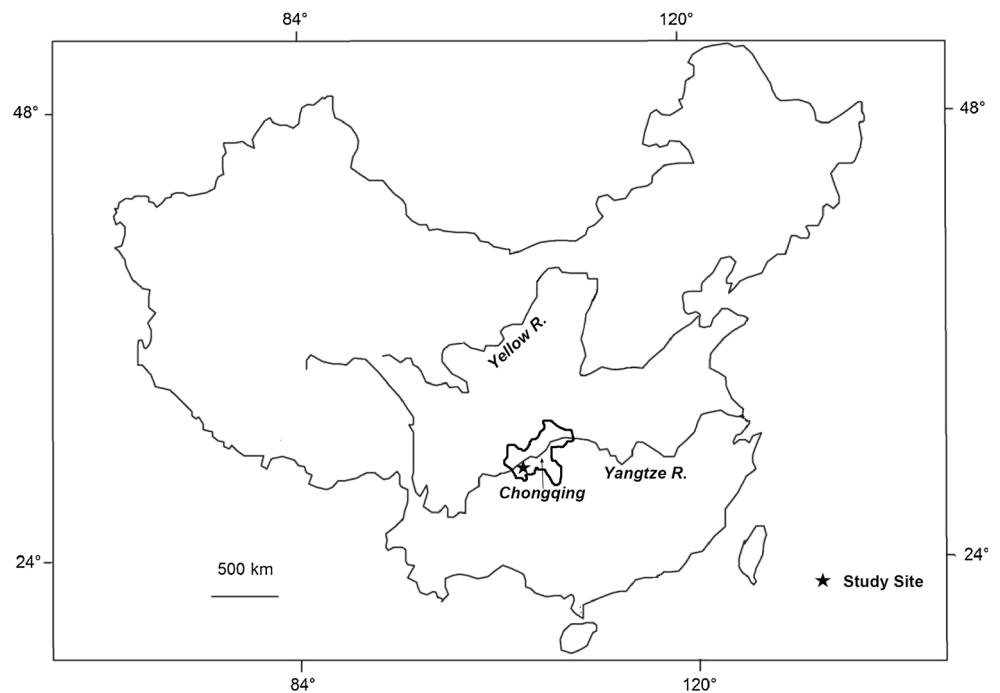
The stability of rock slopes is very important in many engineering works in mountainous areas such as Chongqing, Sichuan and Yunnan in southwest of China. The engineering works include at least the highways, railways, reservoirs and dams. Sandstone and mudstone are two typical sedimentary rocks distributed widely around the world, especially in Chongqing of China. The repeated strata of sandstone and mudstone in Chongqing of China were mainly formed in Upper Triassic, Jurassic and Lower Cretaceous periods (CGMREDC 2002; Wang et al. 2013a). During and after excavation of rock slopes in the repeated strata, the instabilities of cut slopes are very possible. In the present study, a rock slope, excavated in the repeated strata and failed 16.5 years after excavation, was investigated as a case. Time since excavation, as a factor influencing the stability of rock slope, was verified again.

General situation of rock slope and its failure

The rock slope investigated in the present study was excavated in the autumn of 1997 during the period to construct the highway from Tongjiyuanzi to Leishendian in Chongqing of China. The rock cut slope is located at Leishendian Township, near the end of the highway (Fig. 1). The highway was the first highway since Chongqing was upgraded to the fourth municipality directly under the central government of China in March 1997. It was constructed under the financial support from the Asian Development Bank. The total length of the highway was about 87 km, and its width was about 24.5 m with four lanes.

The rock slope was excavated in the repeated strata of sandstone and mudstone formed in Jurassic period. In Chongqing of China, the interbedded deposit of sandstone and mudstone formed in Jurassic period includes six formations with total thickness of about 1944–5230 m (CGMREDC 2002). They are Zhenzhuchong (about 180–320 m in thickness) and Ziliujing (about 300–420 m in thickness) formations formed in Lower Jurassic period, Xintiangu (about 40–490 m in thickness) and Shaximiao (about 1100–2100 m in thickness) formations formed in Middle Jurassic period, and Suining (about 200–600 m in thickness) and Penglaizhen (about 124–1300 m in thickness) formations formed in Upper Jurassic period. The sandstone and mudstone layers of the rock slope were in Shaximiao formation formed in Middle Jurassic period.

Fig. 1 Location of the study site



The maximum height of the rock cut slope was about 40 m, and its slope ratio was 1.0 (vertical direction) to 0.5 (horizontal direction) or its average gradient was about 63.4° . The average dip direction of the rock slope face is about 104° . Because of good rock property and integrity, no reinforcement measure was considered during and after excavation of the rock slope.

At about 8:00 am on April 28, 2014, about 16.5 years after excavation, the rock cut slope failed suddenly (Fig. 2). The rock blocks with about 1000 m^3 in amount, collapsed mainly from the upper sandstone stratum, piled on the highway and blocked the traffic. After about 18 h, at about 2:00 am on April 29, 2014, the rock slope collapsed again (Fig. 3). More rock blocks, with a volume of about 2000 m^3 , collapsed from the upper sandstone stratum too. The diameters of rock blocks were about 0.3–4.5 m. Figure 3 shows that there was an unstable rock mass near the top of slope, which might fall at anytime. In order to avoid possible hazard resulted from the failure of the unstable rock mass, the unstable rock mass, about 300 m^3 in volume, was excavated artificially on April 30, 2014.

Figure 4 shows the full view of the rock slope after excavating the unstable rock mass.

Engineering geological properties

After failure, the engineering geological properties of the rock slope were investigated through engineering geological survey and drilling, geophysical prospecting, laboratory experiments and field tests. Based on topographic survey about 0.05 km^2 in area, nine engineering geological drillings about 357.15 m in total depth and three test trenches, the structure of strata in the rock slope, such as type, distribution, thickness and weathered state, was ascertained. Through tests of longitudinal wave velocity of rock mass in four boreholes about 155.5 m in total testing depth containing 315 testing points, those of longitudinal wave velocity of rock sample in laboratory on 45 rock samples, integrities of rock masses in the rock slope were determined. The values of physical and mechanical properties of rocks were obtained by



Fig. 2 Photograph of rock slope after first failure (on April 28, 2014)



Fig. 3 Photograph of rock slope after second failure (on April 29, 2014)



Fig. 4 Photograph of rock slope after excavation of unstable rock mass (on April 30, 2014)

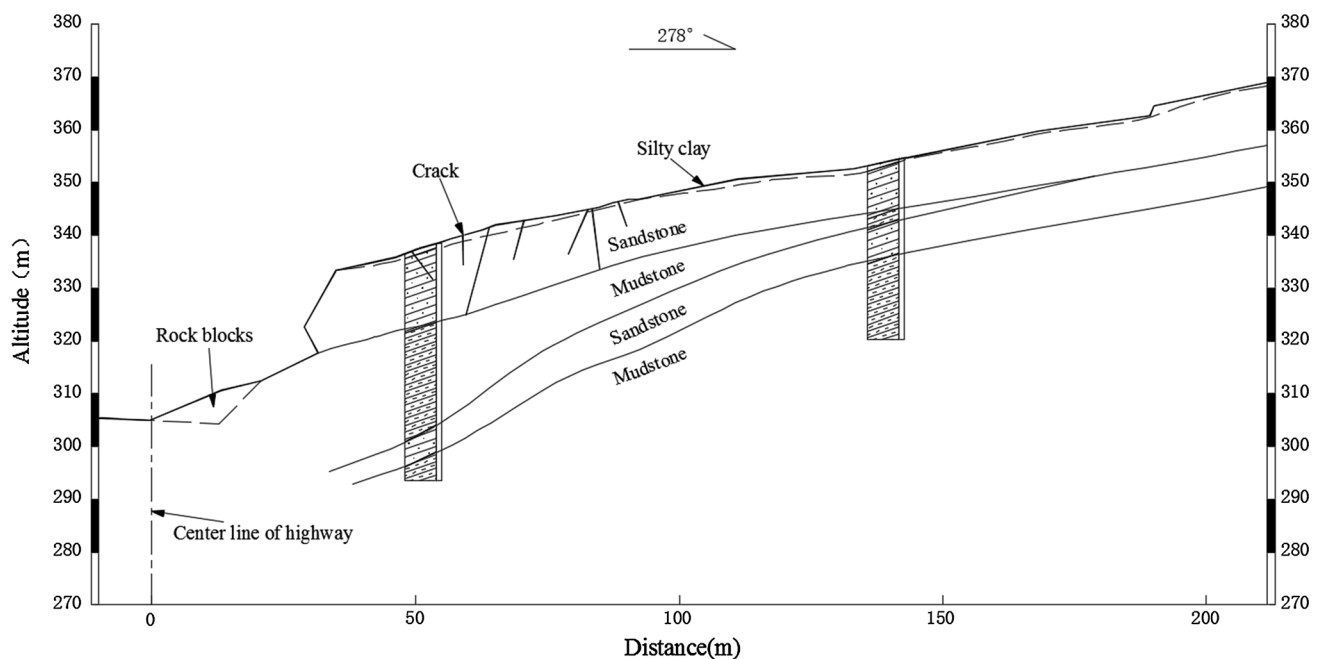


Fig. 5 Typical engineering geologic profile

laboratory experiments on rock samples, such as density test, uniaxial tensile test, uniaxial compressive test and triaxial shear test.

The strata of the rock slope from its top to toe are, respectively, silty clay, sandstone, mudstone, and repeated sandstone and mudstone. The top-layer silty clay formed in Quaternary Holocene period is in plastic state and less than 1.0 m in vertical thickness. The repeated strata of sandstone and mudstone formed in Middle Jurassic period are in moderately weathered state except upper thin layer of 0.5–1.8 m in vertical thickness in highly weathered state. The vertical thicknesses of the sandstone layers are 4.5–19.4 m, and the mudstone layers are 2.3–20.3 m. Figure 5 shows the typical engineering geologic profile.

Physical and mechanical properties of rocks

About 120 rock samples of sandstone and mudstone strata were selected from the cores of the nine engineering geological drillings, and several laboratory experiments on the rock samples were conducted. The laboratory experiments include the density test to determine natural density (ρ) (about 18 samples), uniaxial compressive test to determine natural or saturated uniaxial compressive strength (R_n and R_s) (about 36 samples), uniaxial tensile test to determine natural tensile strength (σ_t) (about 18 samples) and triaxial shear test to determine natural shearing strength (denoted by cohesion (c) and internal frictional angle (φ)) (about 48 samples) of rocks. Based on the test data, the physical and mechanical properties of the sandstone and mudstone were

Table 1 Physical and mechanical properties of rocks

Name of rock	Natural density ρ (g/cm ³)	Natural uniaxial compressive strength R_n (MPa)	Saturated uniaxial compressive strength R_s (MPa)	Softening coefficient K_r	Natural tensile strength σ_t (MPa)	Natural cohesion c (MPa)	Natural internal frictional angle ϕ (°)
Sandstone	2.473 (range 2.450–2.488, 9 samples)	27.383 (range 24.42–29.31, 9 samples)	20.517 (range 18.31–22.24, 9 samples)	0.749	1.442 (range 1.32–1.54, 9 samples)	5.514 (range 5.22–5.88, 24 samples)	44.403 (range 43.81–45.04, 24 samples)
Mudstone	2.526 (range 2.514–2.535, 9 samples)	7.200 (range 6.20–8.22, 9 samples)	4.533 (range 3.81–5.44, 9 samples)	0.630	0.412 (range 0.36–0.50, 9 samples)	1.813 (range 1.71–1.90, 24 samples)	35.152 (range 34.62–35.74, 24 samples)

Table 2 Longitudinal wave velocities of rocks

Depth (m)	Name of rock	Longitudinal wave velocity of rock mass v_{pRM} (m/s)	Longitudinal wave velocity of rock sample v_{pRS} (m/s)	Integrity index of rock mass K_v	Integrity degree of rock mass
1.8–12.1	Sandstone	2852 (range 2697–2991)	3622	0.62	Blocky
12.1–27.2	Mudstone	2710 (range 2626–2839)	3414	0.63	Blocky
27.2–31.6	Sandstone	3014 (range 2925–3132)	3768	0.64	Blocky
31.6–43.0	Mudstone	2844 (range 2531–2959)	3528	0.65	Blocky
43.0–50.0	Sandstone	3169 (range 3019–3258)	3901	0.66	Blocky

measured. The values of ρ , R_n , R_s , σ_t , c and ϕ are listed in Table 1.

The softening coefficient (K_r), as a parameter, is defined as the ratio of R_s to R_n of rock samples selected from the same type and state strata at the same locations. The value of K_r is usually used to evaluate the effects of water on the mechanical properties of rocks. The values of K_r of the sandstone and mudstone in the rock slope investigated in the present study are calculated and listed in Table 1. The value of K_r for the sandstone is 0.749 and that for the mudstone is 0.630. According to Chinese code for investigation of geotechnical engineering (Chinese National Standard, GB 50021-2001, 2009), based on the value of K_r , the rocks can be divided into two types, i.e., softening rock ($K_r \leq 0.75$) and unsoftening rock ($K_r > 0.75$). Thus, both of the sandstone and mudstone in the rock slope belong to the softening rock because of their values of K_r smaller than 0.75.

It is clear from the table that the values of the uniaxial compressive, tensile and shearing strengths of the mudstone are much less than those of the sandstone. The influence of water on the mechanical properties of the mudstone is greater than that of the sandstone, because the value of K_r of the mudstone, 0.630, is smaller than that of the sandstone, 0.749.

Integrities of rock masses

There are usually two methods to evaluate integrity of rock mass. One, which is a qualitative analytical method, evaluates based on features of discontinuities such as size, series, spacing, bonding and type. The other, which is a quantitative

analytical method, evaluates based on value of integrity index of rock mass (K_v). The K_v , as a parameter to evaluate the integrity or discontinuity of rock mass, is defined by the squared value of the ratio of the longitudinal wave velocity of rock mass (v_{pRM}) to that of rock sample selected from the same type and state stratum at the same location. (v_{pRS}), i.e., $K_v = (v_{pRM}/v_{pRS})^2$. According to Chinese code for investigation of geotechnical engineering (Chinese National Standard, GB 50021-2001, 2009), based on the values of K_v , the integrity degrees of rock masses can be divided into five levels, i.e., intact ($K_v > 0.75$), blocky ($0.55 < K_v \leq 0.75$), disturbed ($0.35 < K_v \leq 0.55$), disintegrated ($0.15 < K_v \leq 0.35$) and laminated ($K_v \leq 0.15$).

According to Chinese standard for test methods of engineering rock mass (Chinese National Standard, GB/T 50266-2013, 2013), the value of v_{pRM} should be measured in caves, boreholes or outcrops and that of v_{pRS} measured on rock samples in laboratory. In the present study, the value of v_{pRM} was measured in four boreholes of nine engineering geological drillings and that of v_{pRS} measured by 45 rock samples. The measured values of v_{pRM} and v_{pRS} from a borehole, and the calculated values of K_v for the sandstone and mudstone at different depths, are listed in Table 2. It is clear from the table that the integrity degrees of the sandstone and mudstone layers at different depths are blocky because of the value of K_v in the range of 0.55–0.75.

Structural features

Structural features, such as folds, faults and discontinuities, often control behaviors of rock masses and contribute to

Fig. 6 Typical discontinuities found from slope face in upper sandstone layer

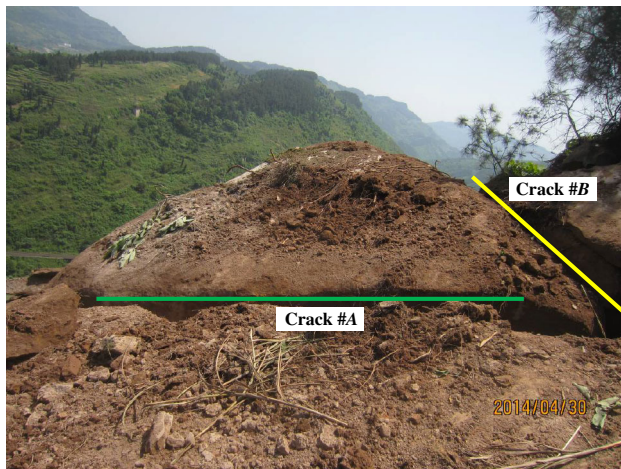
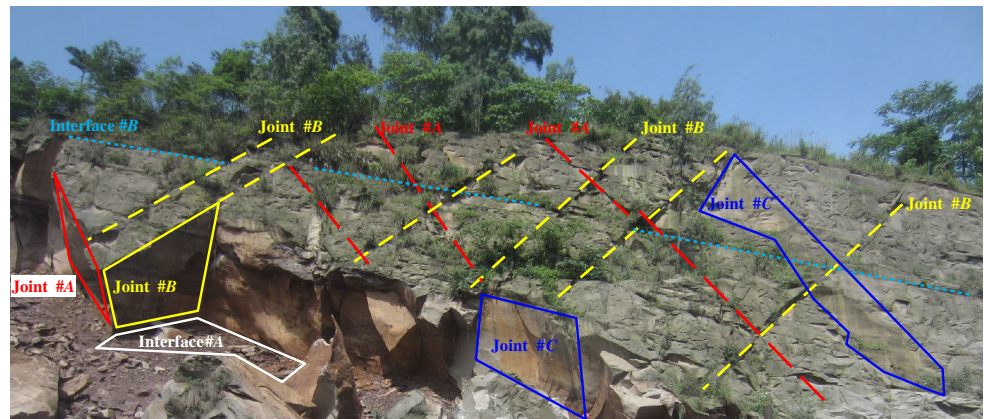


Fig. 7 Typical cracks found from top surface

either the stabilization or destabilization of rock slopes (Brideau et al. 2009; Stead and Wolter 2015). A major cause of slope failures is the presence of unfavorably oriented discontinuities with respect to slope face (Admassu and Shakoor 2013). There are five-set discontinuities in the upper sandstone layer of the rock slope, which can be found from the slope face (Fig. 6). The discontinuities may be divided into two groups. One group is the sedimentary interfaces, which include the interface between the upper sandstone and lower mudstone (“Interface #A” in Fig. 6), and other in the upper sandstone layer (“Interface #B” in Fig. 6). The dip direction and dip of the Interface #A are, respectively, about 72° – 76° and 23° – 40° (median values 74° and 31.5°) and those of the Interface #B about 50° and 23° . The sedimentary interfaces are usually close, and their roughness is rough in small scale, but roughness planar in large scale.

The other group of the discontinuities is the joints, which include three sets, i.e., “Joint #A,” “Joint #B” and “Joint #C” in Fig. 6. The dip direction and dip of the Joint #A are, respectively, about 235° and 66° , those of the Joint

#B about 144° and 64° , and those of the Joint #C about 54° and 61° , respectively. The Joints #A and #B are two-set shear joints. They are close, open or filled with soil particles. Their lengths vary from 2.0 to 20.0 m. Their roughness in small scale is usually smooth, and roughness in large scale near planar. The Joint #C is one-set unloading joints. They are often open and rough, but their lengths are smaller than the Joints #A and #B.

Otherwise, another two-set open and smooth cracks were found on the top surface of the rock slope (“Crack #A” and “Crack #B” in Fig. 7). The dip direction and dip of the Crack #A are, respectively, about 90° and 86° and those of the Crack #B about 160° and 80° . The lengths of the two cracks are about 15.0–25.0 m.

Hydrogeological and groundwater conditions

During the engineering geological survey of the rock cut slope after failure, hydrogeological and groundwater conditions were also investigated. In the maximum depth exposed by the nine geological drillings, about 51.0 m, no groundwater with stable level was found. This means that the groundwater conditions of the rock slope are very simple during non-rainy days. During rainy days, rainwater may flow into the rock slope from the cracks in the upper sandstone layer. Because of low permeability of the lower mudstone layer under the upper sandstone layer, the seeping rainwater may hold or flow in the upper sandstone layer. The seeping rainwater may induce groundwater pressure, soaks rock mass and weakens rocks.

Mechanism of failure

The elapsed time from excavation (in the autumn of 1997) to failure (on April 28, 2014) of the rock slope reaches a length of about 16.5 years. During the period, the rock slope was stable or no failed at least. Why the rock slope

collapsed 16.5 years after excavation? In our opinions, the reasons that resulted in the failure of the rock slope include at least two-aspect factors, i.e., initial excavation and subsequent weathering.

Initial excavation

In the autumn of 1997, in order to construct the highway, the rock slope, with the maximum height 40 m and average gradient 63.4° , was formed by excavation from a natural slope with gradient about 20° (see Fig. 5). During the excavation, the original ground stress balance in the rock masses of the natural slope was broken. New ground stress balance would be reached with the adjustment of stresses and strains in the rock masses during and after excavation. The adjustment of stresses and strains might result in the inducement or expansion of cracks, resulted in the deformation of local rock masses along existing cracks, and even resulted in the looseness of the rock masses. The excavation might increase the discontinuities in the rock slope, and the discontinuities might further accelerate the weathering process of the rock slope after excavation.

In terms of the stress distribution, after excavation, the stress distribution deep in the slope is basically the same as the natural state, while the magnitude of the stress decreased, and stress concentration occurs in many parts of the excavation boundary (Lu et al. 2013). Because of the springback caused by slope excavation, wide range of the tension appears in the slope. In addition, compared with the natural state, the range of concentration of the maximum principal stress in the fracture becomes larger, and the degree of that becomes larger. It is indicated that, after unloading of the slope, the initial stress field has been greatly changed, and more areas which decrease the stability appear in the slope.

The original pressure stresses in the rock masses near the slope face, especially in horizontal or near horizontal directions, might be decreased and even eliminated due to the excavation. The deformation of the rock masses near the upper face of the slope, in particular the deformation toward outside of the slope face, was inevitable during and after excavation. The deformation in the upper sandstone layer of the rock slope might induce tensile stress in the rock masses. If the tensile stress was great enough, the existing cracks were expanded and even new cracks induced. The two-set cracks shown in Fig. 7 might be the cases. The tensioned cracks are usually open and reduce the integrity of the rock masses.

During and after excavation, the shear stress might also be induced in the rock masses, in particular in the upper sandstone layer of the rock slope. The shear stress might result in the deformation of the rock masses along the existing discontinuities or weak planes such as the

interfaces and joints shown in Fig. 6. If the shear stress was great enough, new cracks might be induced.

Subsequent weathering

Typical sedimentary rock masses, such as the sandstone and mudstone of the rock slope investigated in the present study, are usually anisotropic due to bedding planes. The bedding planes or interfaces (see Fig. 6), in the sandstone or mudstone strata and between the strata of sandstone and mudstone, are often weak planes in mechanics. The joints (see Fig. 6) and cracks (see Fig. 7) are another important reason resulting in anisotropy of the rock masses. They are weaker planes in mechanics.

During weathering process, the bedding planes, like other discontinuities such as the joints and cracks, are easily changed or weakened. In general, weathering action includes three dominant processes, i.e., physical, chemical and organic weathering. Physical weathering process results in the disaggregation of rocks without mineralogical change. Chemical weathering process results in the decomposition of the constituent minerals to stable or metastable secondary mineral products. Organic weathering process or biochemical weathering process often includes root wedging, animal activity and microorganisms. The weathering processes on the materials of the sandstone and mudstone formations should be described as mainly physical weathering, combined with chemical and organic weathering on the surface of materials and on the crack and interface walls inside the materials, suggesting all surfaces of materials can be in contact with water. The weathering process is an important factor affecting the stability of slope. Mišcevic and Vlastelica (2014) reported the impact of weathering on slope stability in soft rock mass.

Under the cyclic processes of weathering actions such as drying–wetting, heating–cooling and freezing–thawing, the materials on the slope surface were disintegrated, and the strengths in the interior of the slope also reduced. During the 16.5 years after excavation, the mechanical properties of the sandstone and mudstone should be reduced by the long-term weathering action. Due to the long time of weathering in rock mass, it is difficult to collect the observation data of the rock weathering evolution. Besides, weathering of the rock mass involves many factors, and formula of weathering evolution is hard to be derived. Thus, the research of weathering is mostly based on the data of some experiments (Alavi et al. 2016). The uniaxial compressive strengths of the lightly weathered sandstone and mudstone were investigated by Wang et al. (2013a). The rock samples in the authors' works were also selected from the Shaximiao formation formed in Middle Jurassic period in Chongqing of China but at different locations.

According to the authors' works, the uniaxial compressive strengths of the lightly weathered mudstone were 17.6–25.8 MPa (natural state) and 8.3–15.0 MPa (saturated state) and those of the lightly weathered sandstone 60.0–72.2 MPa (natural state) and 60.0–67.4 MPa (saturated state). Compared to the authors' results, the uniaxial compressive strengths of the moderately weathered sandstone in the present study, 27.383 MPa (natural state) and 20.517 MPa (saturated state) (see Table 1), are only 37.9–45.6% (natural state) and 30.4–34.2% (saturated state) than those of the lightly weathered sandstone in Wang et al. (2013a). The uniaxial compressive strengths of the moderately weathered mudstone, 7.200 MPa (natural state) and 4.533 MPa (saturated state) (see Table 1), are only 27.9–40.9% (natural state) and 30.2–54.6% (saturated state) than those of the lightly weathered mudstone. For the effects on shear strength, results of consecutive weathering and direct shear tests were performed on a siltstone by Yin et al. (2016). Tests showed that at the vertical stress of 50 kPa, a peak stress appears on each curve as shown in Fig. 8, indicating a dilatative feature. As the number of weathering cycle (N) increases, the strength reduction in the material is easily noted as the initial modulus, the peak and the residual strengths decrease. According to test datum, weathering led to strength reduction of 56% for the fresh material at a low stress of 50 kPa, as is usually found in nature where the near surface rock mass is the loosest and weakest (Cetin et al. 2000). It was well known that the weathering process might reduce the mechanical properties of rocks. It was certain that the weathering action during the 16.5 years after excavation was reducing the uniaxial compressive and shear strength of rocks in the rock slope. The properties of sandstone, including mechanical and hydrophysical properties, are very different from those of mudstone (see Table 1). The weathering process of

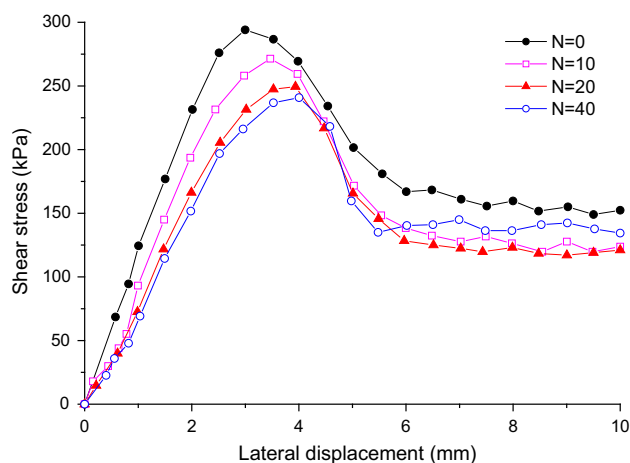


Fig. 8 Relationship of shear stress and lateral displacement for an applied vertical stress of 50 kPa

mudstone is therefore different from that of sandstone. In other words, the weathering rate of mudstone layers in the rock slope is faster than that of sandstone layer. The weathering process of the interface between the upper sandstone and lower mudstone layers may be even faster than the lower mudstone layer (see Fig. 9). Because the mudstone layer under the upper sandstone layer is significantly influenced by weathering, it disintegrates quickly and is removed gradually by the action of gravity and precipitation (Neiman 2009; Admassu et al. 2012). The weathering process results in the reduction of bearing capacity of the lower mudstone layer and may induce the deformation of the upper sandstone layer. The weathering process of rock masses and the induced deformation of the upper sandstone layer may speed up the process of failure of the rock slope.

Water

In generally, water, including groundwater and surface water, is the most important factor in the weathering processes of drying and wetting, freezing and thawing, and also in various chemical and organic weathering processes. Water plays a crucial role in the change of properties of such clayey-dominant rocks as the mudstone in the investigated rock slope (Qiu and Wang 2015). Based on the results of engineering geological survey on the rock slope after failure, no groundwater with stable level was found in the depth of 50 m from the top of slope. During and after rainfall, rainwater flows into the rock slope from the cracks in the upper sandstone layer. Groundwater pressures within the upper sandstone layer may increase suddenly because of the low permeability of the lower mudstone layer under the upper sandstone layer. The rock masses at crack walls may be soaked and weakened, and their mechanic properties deteriorated (see Table 1). The interfaces, such as the

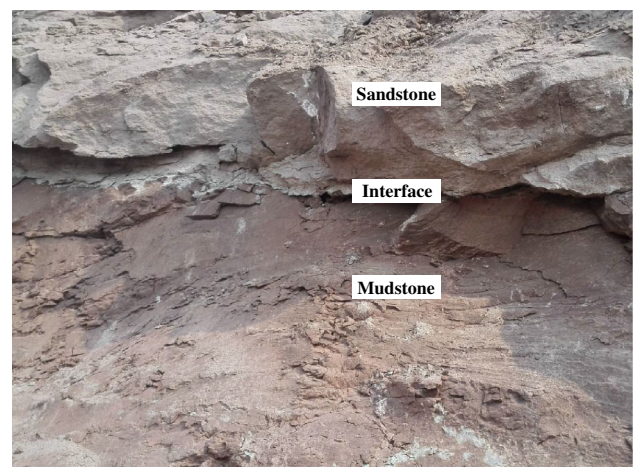


Fig. 9 Interface between upper sandstone and lower mudstone layers

Table 3 Shearing strength of interfaces and argillation intermediate layer

Name of discontinuity	Moisture condition	Cohesion (kPa)	Internal frictional angle (°)
Interface in sandstone	Natural	58.0	23.0
	Saturated	52.0	20.0
Interface in mudstone	Natural	46.0	19.0
	Saturated	38.0	14.0
Interface of mudstone–sandstone	Natural	31.0	13.5
	Saturated	20.0	11.0
Argillation intermediate layer in sandstone	Natural	29.0	15.0
	Saturated	21.0	12.0

interbedded interfaces in the sandstone and mudstone layers, in particular between the upper sandstone and lower mudstone layers, may be wetted and even soaked. The shearing strength of the interfaces may reduce in contact with water. During and after rainfall, the soil particles, in particular fine particles such as clay, silt and sand, entered into open crack with water flow. The soil particles might deposit in the crack, and formed a very thin layer. The thin layer filled in the crack was often called as argillation intermediate layer. Its shearing strength values, especially under saturated conditions, are usually very small.

Table 3 shows the shearing strength values of the interfaces and argillation intermediate layer in sandstone under different moisture conditions. The shearing strength values of the interfaces were determined by composite analysis on the recommended values in Chinese national technical code for building slope engineering (Chinese National Standard, GB 50330-2002, 2002), the empirical values from the engineering experiences in Chongqing of China and the calculated values from the back analysis of the investigated rock slope. The shearing strength values of the argillation intermediate layer were measured by laboratory direct shear tests (ASTM D3080/D3080M-11, 2012). It is clear from Table 3 that the values of shearing strengths of the interfaces are manifestly reduced due to water. The failure time of the rock slope, on April 28, 2014, is exactly in spring rainy season of Chongqing of China. The abundant rainfall in the season may reduce the shearing strength of the interfaces and speeds up the failure of the rock slope.

Although the failure of the rock cut slope took place in spring rainy season, in our opinion, the rainfall wasn't the most important factor inducing the failure because the rainfall in April wasn't the maximum in a year. Based on the climate data of Chongqing of China from 1971 to 2000, average annual rainfall in Chongqing is about 1104.4 mm. Figure 10 shows the distribution of average monthly rainfall from the climate data of Chongqing from 1971 to 2000.

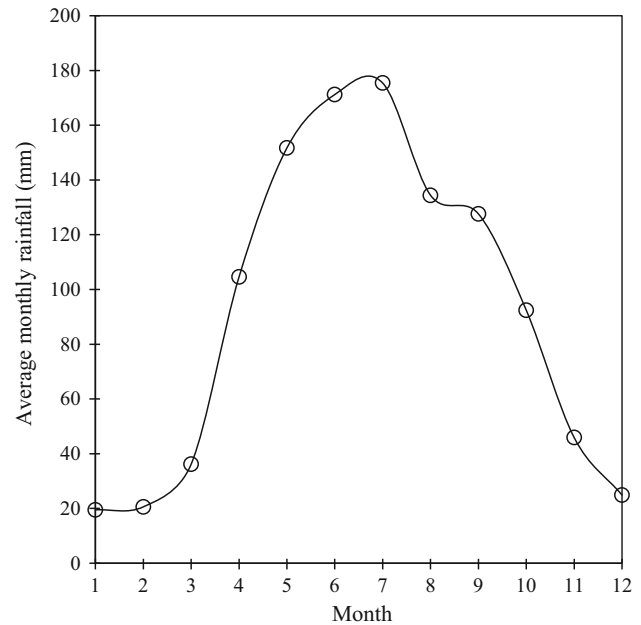


Fig. 10 Distribution of average monthly rainfall in Chongqing from 1971 to 2000

It is clear that the distribution of average monthly rainfall is unbalanced in the term of a year. The month with the maximum of average monthly rainfall is July. The rainfall in April, about 104.6 mm, is only about 60% than that in July, about 175.4 mm.

Stabilization treatment

Stability analysis

The analysis methods on the stabilities of rock slopes are related to their failure modes. The failure modes of rock slopes are controlled by their geologic structures (Stead and Wolter 2015). Typical modes include plane, wedge and toppling failures (Admassu and Shakoor 2013). For extremely weak rock masses or soil-like materials, the failure mode may be circular. For the circular failure mode, the analysis methods on the stability of soil slopes, such as limit equilibrium method, finite element analysis and strength reduction technique (Sarma and Tan 2006; Wang and Lin 2007), are usually applicable. Numerical methods are widely used to evaluate the stability of rock slopes. Typical numerical methods on rock slopes include limit equilibrium analysis (Duncan and Wright 2005), continuum analysis (Grøneng et al. 2010), discontinuum analysis (Stead and Coggan 2012) and brittle fracture and damage analysis (Vyazmensky et al. 2010).

These analysis methods mentioned above, based on the factor of safety (F_s) concept, are called as a deterministic

analysis method, because of its evaluation on stability of a slope based on fixed values of discontinuity parameters (Park and West 2001). In rock slope stability analysis, the uncertainty and variability in geologic conditions and geotechnical parameters, induced by insufficient information concerning the site conditions incomplete understanding or simplification of a failure mechanism, may result in the uncertainty and variability in analysis results. In order to overcome the shortcoming in deterministic analysis method, a probabilistic analysis method on rock slope was developed by Park and West (2001) and Park et al. (2005) and very recently used in back analysis and design of rock slopes by Wang et al. (2013c, d).

Kinematic analysis is a method used to analyze the potential for the various modes of rock slope failures (plane, wedge, toppling failures) that occur due to the presence of unfavorably oriented discontinuities, such as joints, faults, bedding planes, foliation and shear zones that can potentially serve as failure planes (Admassu and Shaikoor 2013). It is performed after plotting discontinuity orientations, slope angle and slope azimuth value on a stereonet. Kinematic analysis is based on Markland's test which is described in Hoek and Bray (1981).

According to the Markland's test, a plane failure is likely to occur when a discontinuity dips in the same direction (within 20°) as the slope face, at an angle gentler than the slope angle but greater than the friction angle along the failure plane (Hoek and Bray 1981). In this failure mode, sliding object overcomes not only the resistance at the bottom, but also the resistance on both sides. In soft rock mass, if the angle of bottom is far greater than that of internal friction, the failure of the rock itself can relieve the side constraint and plane failure will occur. For hard rock, if the top of the slope is cut by discontinuities crosswise and both sides of the rock are separated, plane failure could also occur (see Fig. 11). A wedge failure may occur when the line of intersection of two discontinuities, forming the wedge-shaped block, plunges in the same direction as the slope face and the plunge angle is less than the slope angle but greater than the friction angle along the planes of failure (Hoek and Bray 1981) (see Fig. 11). A toppling failure may result when a steeply dipping discontinuity is parallel to the slope face (within 30°) and dips into it (Hoek and Bray 1981).

In the present study, the kinematic analysis method was used to analyze the possible failure mode of the rock slope in future. Stereonet is circular graph used for plotting planes based on their orientations in terms of dip direction, which are widely used for graphical kinematic analysis. Figure 12 shows the kinematic analysis of the investigated rock slope. It shows that the plane failure along the sandstone–mudstone interface (Interface #A) and the wedge failure along the Joints #B and #C are possible in future. In

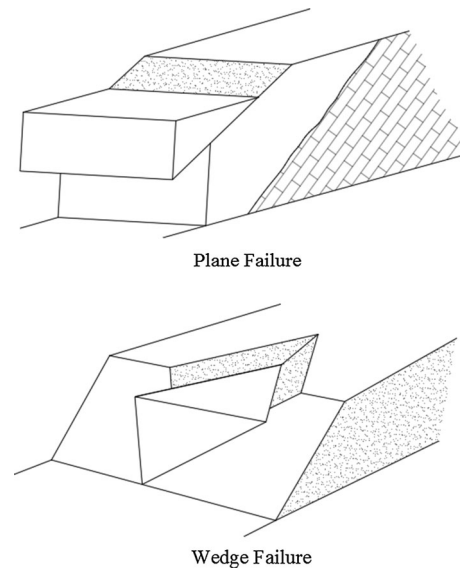


Fig. 11 Planar and wedge failure envelope

order to evaluate the stability of the rock slope, the calculation of the value of F_s , which is defined as the ratio of the resisting forces (shearing strength), that tend to oppose the slope movement, to the driving forces (shear stress), that tend to cause the movement, is necessary. The methods to calculate the values of F_s for the possible plane failure and wedge failure can be found in many books on rock slope engineering (e.g., Hoek and Bray 1981). They can also be obtained by analyzing the equilibrium conditions for the forces acting on the unstable bodies or rock masses.

For the possible plane failure along the Interface #A, the value of F_s is calculated based on the engineering geologic profile shown in Fig. 5 and the shearing strengths of the mudstone–sandstone interface listed in Table 3, and it is 1.15 under natural conditions and 1.04 under saturated conditions. For the possible wedge failure along the Joints #B and #C, the value of F_s is also calculated based on the attitudes of the Joints #B and #C shown in Fig. 12 and the shearing strength of the argillation intermediate layer in the joints of the upper sandstone layer listed in Table 3, and it is 1.09 under saturated conditions. The calculated values of F_s exceed 1.0, and this means that the slope is considered stable. But the calculated values of F_s are very near to 1.0, in particular those under saturated conditions, and this means that the further failure of the rock slope in future is very possible under coming weathering process. For the safety, stabilization to the rock slope should be necessary.

Stabilization

The stabilization of unstable slopes was always paid attention by designers, constructors and investigators in geotechnical engineering works (Justo et al. 2008; Navarro

Fig. 12 Kinematic analysis for failure modes

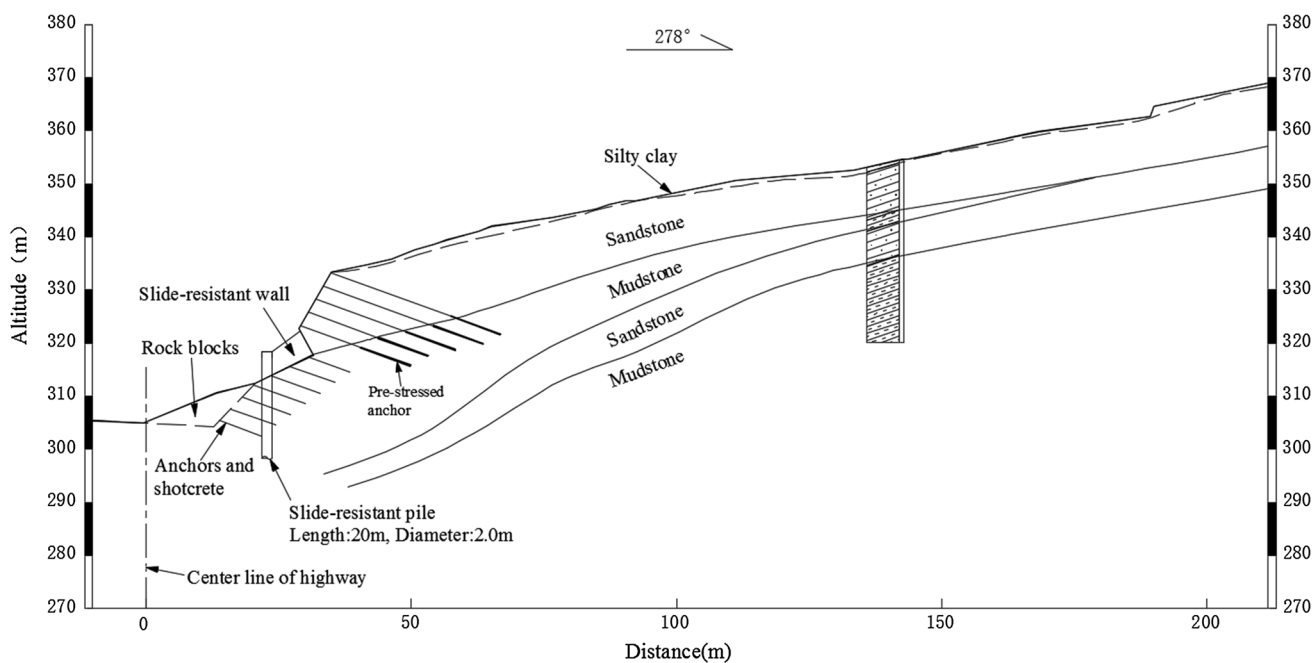
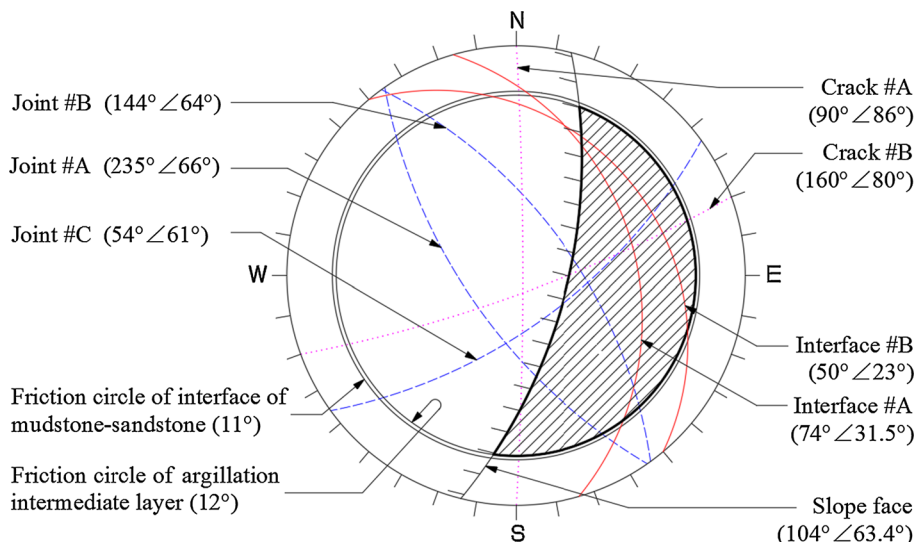


Fig. 13 Stabilization system

et al. 2010; Blanco-Fernandez et al. 2011; Wang et al. 2013d). In general, slope stabilization methods include reducing driving forces, increasing resisting forces or both (Abramson et al. 2002). Excavation of materials from the appropriate part of the unstable ground or drainage of water may reduce driving forces. Construction of retaining structures or other supports can increase resisting forces.

For the rock slope investigated in the present study, an integrated stabilization system was chosen. The integrated stabilization system includes four parts (see Fig. 13). The first part, used to limit the deformation of the upper sandstone layer toward outside, is retaining structures

composed of reinforced concrete piles (slide-resistant pile in Fig. 13) and inclined walls (slide-resistant wall in Fig. 13). The second part, used to increase resisting forces and limit deformation of the upper sandstone rock masses, is pre-stressed anchors. The third part, used to reinforce lower mudstone layer and prevent slope face from further weathering, is anchors and shotcrete panel. The last part, used to increase resisting forces, is interception surface water and drainage groundwater (omitted from Fig. 13).

According to Chinese specifications for design of highway subgrades (Chinese Industry Standard, JTG D30-2015, 2015), the value of F_s after stabilization must reach

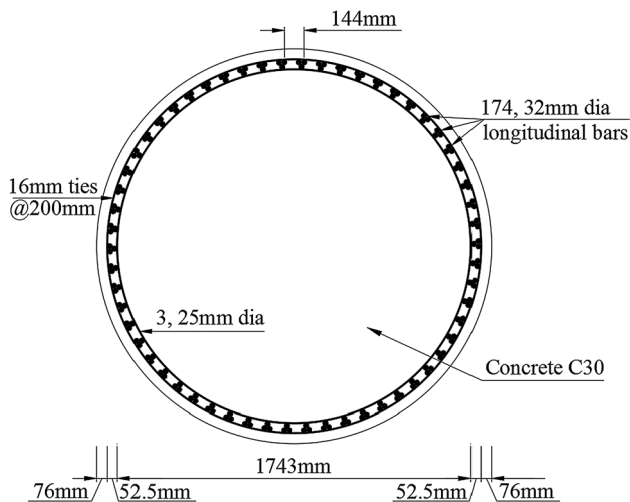


Fig. 14 Reinforced concrete pile cross section

1.20–1.30 in natural states and 1.10–1.20 under saturated conditions. The design methods suggested by Chinese Industry Standard, JTG D30-2015 (2015) and described by Wang et al. (2014) were used to analyze and design the retaining structures and pre-stressed anchors. The anchors and shotcrete panel, interception surface water and drainage groundwater were designed mainly based on the engineering experiences in Chongqing of China.

The retaining structures were composed of vertical slide-resistant piles and inclined slide-resistant walls. Based on analyzing the deformations and stresses of the slide-resistant pile with different pile lengths and section sizes, a row of piles, composed by 12 reinforced concrete piles with 12.0–22.0 m in length (>10.0 m in depth embedded in rock masses) and 2.0 m in diameter, was designed. The center distance between two neighboring piles was 4.0 m. The reinforced concrete pile design with dimensions and reinforcement details is shown in Fig. 14. In order to improve the bearing and deforming behavior of the pile row, a reinforced concrete beam, with 2.0 m in width, 1.0 m in height and 46.0 m in length, was designed to connect the top of piles. Twelve inclined slide-resistant walls, with 2.0 m in thickness, 2.0–5.0 m in height and 6.4–10.0 m in inclined length, were designed to connect the 12 piles and the lower parts of the upper sandstone layer.

About 150 pre-stressed anchors, with 18.0–34.0 m in length (>10.0 m in length of anchorage section), 150 mm in diameter and 25.0° in downward inclination angle, were designed. The center distance between two neighboring anchors in horizontal was about 3.0 m and that in vertical about 2.5 m. The value of pre-stress applied on an anchor was about 400 kN.

The anchors and shotcrete panel were designed to reinforce lower mudstone layer. The dimension of the



Fig. 15 Photograph of rock slope 2 years after stabilization

anchors was about 8.0 m in length, 100 mm in diameter and 2.0 m in center distance in both vertical and horizontal. The size of the shotcrete panel was about 10 cm in thickness and 1200 m² in surface area.

The construction of the stabilization on the rock slope was finished about nearly 2 years ago. Now, the rock slope (Fig. 15) is stable.

Summary and conclusions

As a case on failure of rock slope induced by combination of initial excavation and subsequent weathering, a rock slope, located in the repeated strata of sandstone and mudstone in Chongqing of China and failed about 16.5 years after excavation, was investigated. Based on the above investigations and analyses on its engineering geological properties, failure mode and mechanism, stability analysis and stabilization, the following conclusions may be obtained:

1. The period of rock slope from excavation to collapse may be very long, for instance, about 16.5 years for the rock slope investigated in the present study. Time since excavation, as a factor influencing the stability of rock slopes, should be paid attention in rock slope engineering.
2. Weathering process, especially physical weathering process, is an important factor influencing the long-term stability of rock slope. This is because weathering process may result in disintegration of material on slope surface, and induces reduction of strengths of rocks in the interior of the slope.
3. For rock slopes excavated in sedimentary rocks, like the repeated strata of sandstone and mudstone in the present study, water is a key factor related to the

failure of slopes. This is because water can manifestly reduce the shearing strength of discontinuities.

It is worth mentioning that failures of many rock slopes are sudden or unexpected seemingly, but the deformations of rock masses should be inevitable during the process from stable to unstable and even collapse. The further research works, on the changing process of the deformation of rock masses, stability of rock slope and the relationship between the deformation and stability, may be useful and interesting in rock slope engineering.

Acknowledgements The authors gratefully acknowledge the financial support from the National Science and Technology Support Program of China under Grant No. 2015BAK09B01, the National Natural Science Foundation of China under Grant Nos. 51479012 and 51479014, and the Chongqing Science and Technology Commission of China under Grant No. cstc2015jcyjBX0139, respectively.

References

- Abramson LW, Lee TS, Sharma S, Boyce GM (2002) Slope stability and stabilization methods, 2nd edn. Wiley, New York, pp 462–603
- Admassu Y, Shakoor A (2013) DIPANALYST: a computer program for quantitative kinematic analysis of rock slope failures. *Comput Geosci* 54:196–202
- Admassu Y, Shakoor A, Wells NA (2012) Evaluating selected factors affecting the depth of undercutting in rocks subject to differential weathering. *Eng Geol* 124:1–11
- Alavi SVANK, Tugrul A, Gokceoglu C, Jahed Armaghani D (2016) Characteristics of weathering zones of granitic rocks in Malaysia for geotechnical engineering design. *Eng Geol* 200:94–103
- ASTM D3080/D3080M-11 (2012) Standard test method for direct shear test of soils under consolidated drained conditions. Annual book of ASTM standards. ASTM International, West Conshohocken
- Babiker AFA, Smith CC, Gilbert M, Ashby JP (2014) Non-associative limit analysis of the toppling-sliding failure of rock slopes. *Int J Rock Mech Min Sci* 71:1–11
- Blanco-Fernandez E, Castro-Fresno D, Del Coz Díaz JJ, Lopez-Quijada L (2011) Flexible systems anchored to the ground for slope stabilisation: critical review of existing design methods. *Eng Geol* 122:129–145
- Bouissou S, Darnault R, Chemenda A, Rolland Y (2012) Evolution of gravity-driven rock slope failure and associated fracturing: geological analysis and numerical modeling. *Tectonophysics* 526–529:157–166
- Brain MJ, Rosser NJ, Norman EC, Petley DN (2014) Are microseismic ground displacements a significant geomorphic agent? *Geomorphology* 207:161–173
- Brideau MA, Yan M, Stead D (2009) The role of tectonic damage and brittle rock fracture in the development of large rock slope failures. *Geomorphology* 103:30–49
- Cetin H, Laman M, Ertunc A (2000) Settlement and slaking problems in the world's fourth largest rock-fill dam, the Ataturk Dam in Turkey. *Eng Geol* 56(3–4):225–242
- Chai H-J, Wang J-J, Tang S-C (2009) Study on three-dimensional visualization for rock structure. *Geotech Geol Eng* 27(3):315–323
- Chen Z (1995) Recent developments in slope stability analysis. In: *Proceedings of the 8th international congress ISRM, Tokyo*, pp 1041–1048
- Chigira M, Yokoyama O (2005) Weathering profile of non-welded ignimbrite and the water infiltration behavior within it in relation to the generation of shallow landslides. *Eng Geol* 78(3–4):187–207
- Chinese Industry Standard, JTG D30-2015 (2015) Specifications for design of highway subgrades. Ministry of Transport of the People's Republic of China (**in Chinese**)
- Chinese National Standard, GB 50021-2001 (2009) Code for investigation of geotechnical engineering. The Ministry of Housing and Urban-Rural Development of P. R. China, Beijing, P. R. China (**in Chinese**)
- Chinese National Standard, GB 50330—2002 (2002) Technical code for building slope engineering. The Ministry of Housing and Urban-Rural Development of P. R. China, Beijing, P. R. China (**in Chinese**)
- Chinese National Standard, GB/T 50266-2013 (2013) Standard for test methods of engineering rock mass. The Ministry of Housing and Urban-Rural Development of P. R. China, Beijing, P. R. China (**in Chinese**)
- Chongqing Geological and Mineral Resource Exploration and Development Corporation (CGMREDC) (2002) Chongqing geological map (scale 1:500000). Chongqing Yangtze River Map Printing Factory, Chongqing
- Duncan JM, Wright SG (2005) Soil strength and slope stability. Wiley, Hoboken
- Duperret A, Taibi S, Mortimore RN, Daigneault M (2005) Effect of groundwater and sea weathering cycles on the strength of chalk rock from unstable coastal cliffs of NW France. *Eng Geol* 78(3–4):321–343
- Glastonbury J, Fell R (2000) Report on the analysis of “rapid” natural rock slope failures. University of New South Wales. School of Civil and Environmental Engineering report no. R390
- Grøneng G, Lu M, Nilsen B, Jenssen AK (2010) Modelling of time-dependent behaviour of the basal sliding surface of the Åknes rockslide area in western Norway. *Eng Geol* 114:412–422
- Hack R, Price D, Rengers N (2003) A new approach to rock slope stability—a probability classification (SSPC). *Bull Eng Geol Environ* 62:167–184
- Hall K, André MF (2001) New insights into rock weathering from high-frequency rock temperature data: an Antarctic study of weathering by thermal stress. *Geomorphology* 41:23–35
- Hoek E, Bray JW (1981) Rock slope engineering. Institute of Mining and Metallurgy, London
- Justo JL, Azañón JM, Azor A, Saura J, Durand P, Villalobos M, Morales A, Justo E (2008) Neotectonics and slope stabilization at the Alhambra, Granada, Spain. *Eng Geol* 100:101–119
- Liu YC, Chen CS (2007) A new approach for application of rock mass classification on rock slope stability assessment. *Eng Geol* 89:129–143
- Lu W, Hu Y, Yang J, Chen M, Yan P (2013) Spatial distribution of excavation induced damage zone of high rock slope. *Int J Rock Mech Min Sci* 64:181–191
- Mišćević P, Vlastelica G (2014) Impact of weathering on slope stability in soft rock mass. *J Rock Mech Geotech Eng* 6:240–250
- Moore JR, Gischig V, Burjanek J, Löw S, Fäh D (2011) Site effects in unstable rock slopes: dynamic behaviour of the Randa instability (Switzerland). *Bull Seismol Soc Am* 101:3110–3116
- Navarro V, Yustres A, Candel M, López J, Castillo E (2010) Sensitivity analysis applied to slope stabilization at failure. *Comput Geotech* 37:837–845
- Neiman W (2009) Lessons learned from rates of mudrock undercutting measured over two time periods. *Environ Eng Geosci* 15(3):117–131
- Nicholson DT (2004) Hazard assessment for progressive, weathering-related breakdown of excavated rock slopes. *Q J Eng Geol Hydrogeol* 37:327–346

- Pantelidis L (2009) Rock slope stability assessment through rock mass classification systems. *Int J Rock Mech Min Sci* 46:315–325
- Park H, West TR (2001) Development of a probabilistic approach for rock wedge failure. *Eng Geol* 59:233–251
- Park H-J, West TR, Woo I (2005) Probabilistic analysis of rock slope stability and random properties of discontinuity parameters, Interstate Highway 40, Western North Carolina, USA. *Eng Geol* 79:230–250
- Qiu Z-F, Wang J-J (2015) Experimental study on the anisotropic hydraulic conductivity of a sandstone–mudstone particle mixture. *J Hydrol Eng ASCE* 20(11):04015029
- Romana M (1985) New adjustment ratings for application of Bieniawski classification to slopes. In: Proceedings of the international symposium on role of rock mechanics, Zacatecas, Mexico, pp 49–53
- Sarma SK, Tan D (2006) Determination of critical slip surface in slope analysis. *Géotechnique* 56(8):539–550
- Stead D, Coggan JS (2012) Numerical modelling of rock-slope instability. In: Clague JJ, Stead D (eds) *Landslides: types, mechanisms and modelling*. Cambridge University Press, New York, pp 144–158
- Stead D, Wolter A (2015) A critical review of rock slope failure mechanisms: the importance of structural geology. *J Struct Geol* 74:1–23
- Stead D, Eberhardt E, Coggan JS (2006) Developments in the characterization of complex rock slope deformation and failure using numerical modelling techniques. *Eng Geol* 83:217–235
- Vyazmensky A, Stead D, Elmo D, Moss A (2010) Numerical analysis of block caving-induced instability in large open pit slopes: a finite element/discrete element approach. *Rock Mech Rock Eng* 43:21–39
- Wang J-J, Lin X (2007) Discussion on “Determination of critical slip surface in slope analysis”. *Géotechnique* 57(5):481–482
- Wang J-J, Chai H-J, Li H-P, Zhu J-G (2008) Factors resulting in the instability of a 57.5 m high cut slope. In: Proceedings of the 10th international symposium on landslides and engineered slopes, pp 1799–1804
- Wang J-J, Zhang H-P, Deng D-P, Liu M-W (2013a) Effects of mudstone particle content on compaction behavior and particle crushing of a crushed sandstone–mudstone particle mixture. *Eng Geol* 167:1–5
- Wang J-J, Zhao D, Liang Y, Wen H-B (2013b) Angle of repose of landslide debris deposits induced by 2008 Sichuan Earthquake. *Eng Geol* 156:103–110
- Wang L, Hwang JH, Juang CH, Atamturktur S (2013c) Reliability-based design of rock slopes—a new perspective on design robustness. *Eng Geol* 154:56–63
- Wang L, Hwang JH, Luo Z, Juang CH, Xiao J (2013d) Probabilistic back analysis of slope failure—a case study in Taiwan. *Comput Geotech* 51:12–23
- Wang J-J, Liang Y, Zhang H-P, Wu Y, Lin X (2014) A loess landslide induced by excavation and rainfall. *Landslides* 11(1):141–152
- Yin Y, Zhang BY, Zhang JH, Sun GL (2016) Effect of densification on shear strength behavior of argillaceous siltstone subjected to variations in weathering-related physical and mechanical conditions. *Eng Geol* 208:63–68