Failure mechanism and stabilization of a basalt rock slide with weak layers

Demin Xuea, Tianbin Lia, Shuai Zhangb, Chunchi Ma, Meiben Gao, Ji Liu

a State Key Laboratory of Geohazard Prevention and Geoenvironment Protection, Chengdu University of Technology, Chengdu 610059, Sichuan, China
b MOE Key Laboratory of Soft Soils and Geoenvironmental Engineering, Institute of Civil Engineering, Zhejiang University, Hangzhou 310058, China

ARTICLE INFO

Keywords:
- Excavation
- Landslide
- Finite element analysis
- Slope stability
- Limit equilibrium analysis
- Pile

ABSTRACT

Multiple slides of rock slopes with dip weak layers have occasionally occurred in Chinese roadway engineering, representing a very complex problem associated with roadway excavation. Unfortunately, the failure mechanisms of such slopes are not clearly investigated prior to excavation, thus leading to expensive remedial work installed after the significant deformative phenomena occurred. Furthermore, pre-reinforced piles have not received enough attention to be used to stabilize the slopes during the excavation process, which enables the slope movements. This paper, therefore, aims to explore the failure mechanisms of a basalt rock slide with dip weak layers and analyse the impacts of pre-reinforced piles on the slope stability. Field investigations, laboratory tests, and in situ deformation monitoring are conducted. Slope stability before and after excavation is analyzed using the finite element and limit equilibrium analysis methods, respectively. The outcomes of the analyses reveal that excavation and the presence of dip weak layers are the main contributory factors to the rock slide. The failure surface of the slope varies with increases in excavation rather than always occurring along the revealed weak layers. The instability appears to be multiple translational slides. Additionally, a finite element analysis for the slope pre-reinforced with row piles during different stages of excavation is conducted. Based on the study results, the pre-reinforced pile is crucial for the slope stability, and the impact of pile position should be prudently considered in design.

1. Introduction

The presence of a weak layer usually plays an adverse role in slope stability, resulting in inducement or acceleration of slope failure (Alhomoud and Tubeileh, 1998; Xu et al., 2013). Most previous studies focused on the stability and failure mechanisms of slopes with dip weak layers that moved along a single sliding surface. In particular, Zhu et al. (2010) suggested that slope failure will easily occur along potential sliding surfaces because the shear strength of weak surfaces is significantly low. Deng et al. (2011) proposed a displacement calculation method simulating the failure of a dip slope with a thin sandy layer, triggered by the Niigata-ken Chuetsu earthquake in 2004. Huang et al. (2013, 2017) proposed a rotational-translational mechanism to evaluate the stability of slopes with a weak layer and pile using the upper bound limit analysis. Yao et al. (2014) proposed a simplex-finite stochastic tracking method to target the position of a potential sliding surface in a soft rock slope containing weak structural planes and discontinuities. Rodríguez-Ochoa et al. (2015) quantified the influence of weak layers on earthquake-induced submarine slope failures. However, studies on multi-slides of slopes with weak layers, with particular focus on excavation, are rarely reported. As is well known, a slope is generally expected to slide along a dip weak layer after the weak layer is revealed by excavation. This demands that pre-reinforced piles be installed prior to excavation to prevent the potential landslide movement, thereby ensuring the slope safety. Unfortunately, there are very few well-documented case studies in the literatures considering the impacts of pre-reinforced piles on the stability of slopes with dip weak layers associated with excavation.

The present case study concerns a basalt rock slide that poses a significant threat to the safety of the Bi-Wei highway construction. When the highway excavation was completed, some downslope movements were observed on the slope, including large U-shaped scarps detected in the rear of the landslide, surface drain distortions, and a sliding surface outcropped at the toe (Fig. 1). To provide warning references to engineers for landslide hazard mitigation, an analysis of the possible evolution and mechanisms of slope failure was urgently needed. A field investigation was immediately conducted to develop a suitable slope stabilization plan. In situ soil samples were taken from the slope and tested in the laboratory to obtain geotechnical parameters. Three continuous coring boreholes equipped with full length inclinometer tubes were drilled in the slope to explore the potential sliding surface of the landslide. In this study, the results of the
laboratory tests, deformation monitoring, finite element analysis, and limit equilibrium analysis are utilized to explore the failure mechanisms of the basalt rock slide. Finally, the impacts of pre-reinforced piles on the slope stability associated with excavation are investigated through finite element analyses. The analysis results can provide a beneficial reference for the design of a dip rock slope to prevent the occurrence of rock slide.

2. Geological settings

The study area is located at mileage K103 of the Bi-Wei Highway in Guizhou, in southwest China (Fig. 2). The slopes and elevations of the area generally are in the ranges of 20° to 43° and 1760 m to 1920 m, respectively. The study area is located in a seismically active region of China. A Yadu fault zone distributed along the study area (Fig. 3) has little effect on regional stability and should not be able to produce an earthquake of a magnitude greater than 6.0. Geologically, the study area consists of Quaternary-Holocene eluvium and Upper Permian basalt. Basalt underlies the eluvium and appears to be completely or highly weathered. Weak layers with dips of 20–40° are well developed along the bedding surface, which contains 50–150 mm of clay gouge (Fig. 4). Therefore, the characteristics of the bedrock itself may be advantageous for slope instability in this area.

3. Investigation methodology

3.1. Field investigations

The investigated slope was a cut slope having four steps connected with a bridleway 3 m in width (Figs. 5 and 6). Trial pits and borehole investigations revealed that the slope was composed of eluvium deposits and basalt. The bedrock is highly and moderately weathered basalt. According to the latest quantitative GSI chart modified by Sonmez and Ulusay (1999), basalt at the slope surface occurs as a very blocky rock mass (Fig. 7a). Two reddish-brown weak layers labelled layers 1 and 2 were identified at elevations of 1807 m and 1821 m, respectively. An approximately 60–80 mm width of clay gouge was observed among them (Fig. 7b). Both weak layers have an attitude of 172°±25° and dip out of the slope at approximately 17°. As revealed in Fig. 6, weak layers are considered to form the basement of the landslide. No surface springs or groundwater tables were observed in the slope. Therefore, groundwater could not contribute to the failure of the slope.

3.2. Inclinometer measurements

To analyse the stability of the basalt rock slope when excavation was performed, three continuous coring boreholes (ZK-1, ZK-2, and ZK-3) with full length inclinometer tubes were drilled in various sections of the slope (Fig. 6); the lengths of ZK-1, ZK-2, and ZK-3 were recorded as 30, 30, and 37.5 m, respectively. Inclinometer measurements were conducted on September 9, 2011. The inclinometer profiles are shown in Fig. 8. It can be seen that the basalt slope was still undergoing noticeable displacements after full excavation. The landslide movement was progressive. For inclinometers I1 and I3, the cumulative displacement curve displayed an L-shape, with a single typical sliding surface on the curve. The sliding surfaces were located at 14 and 20 m depths, coinciding with the depth of weak layer 1; the cumulative displacements at different depths were considered to be approximately equivalent, indicating that the sliding mass (where boreholes ZK-1 and ZK-3 were located) moved as a whole along the sliding surface. A saw-tooth-shaped cumulative displacement curve was observed for inclinometer I2. The sliding surface was located at a depth approximately 8 to 21 m below the ground surface. Note that the cumulative displacements at different depths were not uniform. Apparent increases in deformation were found at an 11 m depth (e.g., the position of weak layer 1), and a 19 m depth, which is very close to the position of weak layer 2. In view of the above monitoring results, the upper two inclinometers I1 and I3 showed a clearly translational slide along weak layer 1. However, the lowest inclinometer, I2, showed different
behaviour along the two weak layers, which may indicate the occurrence of multiple slides.

4. Direct shear tests on weak layer

The results of field investigations and inclinometer measurements indicate that the basalt rock slide occurred along the two weak layers. The slope stability therefore depends on the following shear strength parameters for the weak layers: cohesion (c) and friction angle (Φ) (Hatzor and Levin, 1997; Xie et al., 2006; Ghazvinian and Taghichian, 2010; Zhu et al., 2010; Xu et al., 2013). In this study, the highly weathered basalt tended to be easily disintegrated. It was difficult to obtain a fully qualified specimen with a regular shape during field sampling. Therefore, traditional direct shear tests are not suitable for this study because a regular specimen cannot be provided. This test used a new shear instrument developed by the State Key Laboratory of Geological Disaster Prevention at the Chengdu University of Technology: the XJ-2 portable shear instrument. This instrument was used to test the shear strength of rock samples with irregular shapes. Because regular geometry is not required for these specimens, this test equipment is applicable to rock samples taken from the structural surface of a landslide (e.g., bedding layers, fault fracture zones, sliding surfaces, weak layers, shale, phyllite, schist, and highly weathered rock). Initially, the irregular samples are wrapped in concrete and cast into specimens with fixed sizes and shapes. The direct shear test can be conducted after the regular shaped specimens are prepared and installed in a specific instrument with same size (i.e., a portable shear instrument). Shearing tests and data compilation can then be performed according to the Mohr-Coulomb formula.

Rock samples from weak layers were taken from the investigated
Fig. 4. Exposed weak layers within the highly weathered basalt in the study area.

Fig. 5. Overview of excavated basalt slope.

Fig. 6. Geological model of excavated basalt slope.
slopesite. The results of the direct shear tests for samples are shown graphically in Fig. 9. The best linear fit results indicate that the undisturbed weak layer has a cohesion value ($c$) of 30.6 kPa and a friction angle ($\Phi$) of 17°. The typical shear failure of the rock samples from the weak layer is determined using the test (Fig. 10); the results demonstrate that the weak layer could act as a potential sliding surface under gravitational force.

5. FEM slope stability analysis before and after excavation

To evaluate the stability of the basalt slope before and after excavation, the major geologic profile of the slope (Fig. 6) is investigated. A finite element analysis is performed to explore failure mechanisms related to excavation, incorporating a stability analysis into a finite element stress-based method using Sigma-W and Slope-W (GEO-SLOPE International Ltd., 2007). In the finite element analysis, the evolution of the landslide could also be highlighted using Sigma-W. Sigma-W stresses are used by Slope-W to calculate the safety factors of the slope. Determining the position of the critical failure surface with the lowest factor of safety remains one of the key issues in a stability analysis. There are many different ways for defining the shape and positions of trial failure surfaces in SLOPE/W. In this case study, the slope material includes two weak layers, and their strength is obviously less than the surrounding basalt; according to experience, the sliding surface is more likely to occur along the two weak layers. Therefore, the critical failure surface is generated with Block Search by specifying two grids of points placed along the weak layers. The searched critical failure surface in SLOPE/W is compared with the positions of the maximum shear strain in Sigma-W to determine its accuracy. As is well-known, the Mohr-Coulomb failure criterion, commonly used in the geotechnical engineering, shows that the failure of the rock and soil mass is caused by the reach of the shear stress on a surface to its shear strength; thus, a large shear deformation must have occurred on the shear failure surface. Therefore, the failure surface can be determined through searching the positions of maximum shear strain (Cheng et al., 2007; Li et al., 2013). For plane strain conditions, the maximum shear strain $\gamma_{\text{max}}$ can be defined as

$$\frac{\gamma_{\text{max}}}{2} = \sqrt{\left(\epsilon_x - \epsilon_y\right)^2 + \left(\gamma_{xy}\right)^2}$$  \hspace{1cm} (1)

where $\epsilon_x$ and $\epsilon_y$ are the normal strains in the $x$ and $y$ directions, respectively, and $\gamma_{xy}$ is the shear strain on the $xy$ plane.

As listed in Table 1, the input parameters are determined based on laboratory tests, literature values (Shen et al., 2006; Xue et al., 2015), and empirical engineering experiences. Fig. 11 shows the finite element mesh for the original slope along with the materials that are assumed to obey the Mohr-Coulomb failure criterion in the analysis. The left and right-side boundaries of the FEM model were specified as no-horizontal-displacement boundaries, and the bottom boundary was set as a no-horizontal and no-vertical-displacement boundary. The excavation process was modelled in Sigma-W using excavation elements that can be applied at various stages of the analysis; the piles and anchors are modelled in Sigma-W using structural beam elements and structural bar elements, respectively. The initial stresses in the FEM model are gravity stresses. The simulation begins with the original slope, and subsequent unloading and reinforcement is undertaken in four steps.

The FEM analysis of basalt slope was performed to obtain the characteristic features of the slope failure during excavation progress. Fig. 12 shows the progressive evolution nephograms of the maximum shear strains within the slope during different stages of excavation, and the corresponding factor of safety $Fs$ for the slope. As can be observed, the excavation significantly increases the maximum shear strains mainly in the two weak layers and leads to a progressive failure within the slope. The critical failure surfaces are very consistent with the position of the maximum shear strains, indicating the critical failure surfaces are accurate. The calculated factor of safety $Fs$ for the original slope was 1.241 (Fig. 12a), i.e., the slope is in a stable state prior to excavation. When the first staged excavation reveals weak layer 1, $Fs$ for the excavated slope is 1.051, and the slope appears to move along weak layer 1 (Fig. 12b). The slope stability is continuously decreased with increasing excavation, and the maximum shear strains at toe propagate upward along weak layer 2 (Fig. 12c). Note that the stress redistribution caused by the third staged excavation significantly increases the maximum shear strains; two clearly defined failure surfaces, C1 and S1, are eventually formed along the two weak layers in the slope (Fig. 12d), and the corresponding $Fs$ are 1.027 and 1.039, respectively.

Finally, the slope is reinforced with piles and anchors, and its $Fs$ reaches 1.239 (Fig. 12e).

The horizontal displacements of basalt slope at the position of the inclinometer with the staged excavations by FEM analysis is illustrated in Fig. 13. The figure shows the horizontal displacements significantly increase as the excavation proceeds. Two major increases in the horizontal displacement are recorded; these displacements are clearly
triggered by excavation and weak layers, indicating that excavation and weak layers are the primary factors affecting the slope behaviour. Note that two jumps of the displacement along the depth at the position of the inclinometer I2, indicating the occurrence of the two multi-slides. As expected, the slope initially moves along weak layer 1 after the first and second staged excavation, and then potentially slides along the two weak layers. The outcomes obtained from the FEM analysis are in good agreement with the inclinometer measurement results, although the final FEM displacement values are slightly smaller than the measured values, as shown in Fig. 13.

Based on the above FEM analysis, it can be concluded that 1) the stability of the slope continuously decreases with increasing excavation, 2) the position of the potential failure surface varies with increasing excavation, 3) two multi-slides are ultimately developed along the two potential failure surfaces (S1 and C1), and 4) pile and anchors improve the stability of the excavated slope.

6. Limit equilibrium stability analysis of the excavated slope

Based on the inclinometer measurement results, the sliding surfaces where the upper two inclinometers I1 and I3 were located were detected to occur at depths of 14 and 20 m below the ground surface, respectively, coinciding with the depth of weak layer 1. However, the depth of the sliding surface where the lowest inclinometer I2 was located ranged from 8 to 21 m, which must be clearly determined to conduct the LEM stability analysis. As demonstrated by the FEM analysis, two potential sliding surfaces (C1 and S1) developed along the two weak layers; their positions are very consistent with the inclinometer measurement results. The actual sliding surfaces are therefore determined to be C1 and S1.

A limit equilibrium stability analysis of the excavated slope is conducted using the Slope-W software. Morgenstern-Price slice methods are applied to analyse the slope stability. The Fs values for sliding
surfaces C1 and S1 are calculated to be 1.035 and 1.056, respectively. Both of these values are slightly larger than those calculated in the FEM analysis. Deformation of the excavated slope is therefore caused both by excavation and by the two weak layers, although the Fs values are slightly larger than 1.0.

7. Failure mechanism of the slope considering excavation

Based on the field investigation results, FEM analysis, and LEM analysis, the failure mechanism of the basalt rock slide can be described as follows:

1) Excavation is the main contributory factor to the basalt rock slide. Inappropriate excavation decreased the slope resistance and created a free surface for potential sliding. Moreover, excavation revealed two weak layers. As a result, slope stability sharply decreased, and a significant progressive failure of basalt slope occurred.

2) The two weak layers are the fundamental cause of the basalt rock slide. Failure initially occurred along weak layer 1 because of its low resistance capacity and was continuously exaggerated along the two weak layers during the excavation process. Two fully defined failure surfaces were ultimately well developed along the two weak layers after excavation revealed weak layer 2.

3) A global slide and local slide are found to occur in the slope after full excavation. Multiple rock translational slides are used to define the failure mechanism: the global sliding of basalt rock masses on an inclined planar failure surface consisting of several planes (i.e., the two weak layers). Meanwhile, local sliding is kinematically possible only if it is accompanied by the significant internal distortion of the moving mass. This was demonstrated in the measurements of inclinometer I2, which recorded two apparent increases in displacement; the displacement along sliding surface C1 was significantly larger than that along sliding surface S1. The basal segment of the failure surface often follows the weak layer. The head of the sliding mass separates from the stable rock mass along a deep tension crack or scarp.

8. Stabilization works of the excavated slope

Piles and anchors have proven to be an effective countermeasure for improving the stability of a slope (Song et al., 2012; Lirer, 2012; Topsakal and Topal, 2015). In this case study, one row of piles and three rows of anchors were constructed to reinforce the excavated slope, which experienced significant deformation because of being cut for highway construction. One row of piles (pile row 1) with dimensions of 2 × 3 × 18 m (width × height × length), spaced 6 m apart, was
installed at the toe of the excavated slope. Active pressure-grouted anchors, with angles of 25° from the horizontal and lengths of 34–38.5 m were installed at intervals of 3 m along the second-grading slope surface. All piles and anchors were deep enough to reach stable bedrock, as shown in Fig. 6. Further FEM stability analysis of the excavated slope is conducted using the Sigma-W and Slope-W module, which takes into account the installed pile and anchors. The searched critical failure surface is shown in Fig. 12e. The final $F_s$ value is calculated to be 1.239. Note that the slope actually moved along the two pre-existing sliding surfaces $C_1$ and $S_1$, as demonstrated both by the inclinometer measurement results and by the FEM calculated results; therefore, it is necessary to obtain their respective values of $F_s$. By calculation, the $F_s$ value is 1.262 and 1.369 for $C_1$ and $S_1$, respectively, as shown in Fig. 12e. In accordance with the Chinese specifications for design of highway subgrades JTGD30-2015 (Ministry of Transport of the People’s Republic of China, 2015), the required safety factor of the excavated slopes is 1.2 to 1.3 for permanent roadway operation safety. All of $F_s$ for the critical failure surface and two pre-existing failure surfaces are greater than 1.2, indicating the piles and anchors successfully stabilize the sliding mass and guarantee the slope safety, as indicated by the fact that the deformation rate gradually decreased and reached zero after the construction of piles and anchors (Fig. 14).

### Table 1

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Superficial deluvium</th>
<th>Weak layer</th>
<th>Highly weathered basalt</th>
<th>Moderately weathered basalt</th>
<th>Pile 1,2,3</th>
<th>Anchor</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\gamma$ (kN/m$^3$)</td>
<td>20.5</td>
<td>19</td>
<td>23</td>
<td>28</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>$C$ (kPa)</td>
<td>23</td>
<td>30.6</td>
<td>25</td>
<td>1300</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>$\phi$ (°)</td>
<td>22</td>
<td>17</td>
<td>28</td>
<td>45</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>$E$ (kPa)</td>
<td>$7 \times 10^5$</td>
<td>$5 \times 10^5$</td>
<td>$11 \times 10^5$</td>
<td>$75.8 \times 10^5$</td>
<td>310$^5$ (*)</td>
<td>1.95$^5$ (*)</td>
</tr>
<tr>
<td>$\theta$</td>
<td>0.3</td>
<td>0.37</td>
<td>0.33</td>
<td>0.27</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>$\psi$ (°)</td>
<td>0</td>
<td>0</td>
<td>24</td>
<td>34</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>$\sigma_t$ (kPa)</td>
<td>0</td>
<td>0</td>
<td>350 (*)</td>
<td>3415 (*)</td>
<td>28$^5$ (*)</td>
<td>186$^5$ (*)</td>
</tr>
<tr>
<td>Moment of inertia (m$^4$)</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>4.5</td>
<td>–</td>
</tr>
</tbody>
</table>

(*) the literature values.

9. Discussion

Weak layers combined with excavation always decrease the stability of a dip rock slope. Consequently, remediation measures, such as piles and anchors, must be built to stabilize such slopes, as described in this case study. Avoiding rock slides is a key problem in engineering practice. Much more attention is therefore paid to pre-reinforced measures. In this case study, a series of FEM trials is completed for a basalt slope pre-reinforced with piles and is used to evaluate their respective stabilizing effectiveness. Pile row 1 has dimensions of $2 \times 3 \times 35$ m (width × height × length), and the piles are spaced 6 m apart. Pile row 2 has dimensions of $2 \times 3 \times 25$ m (width × height × length), and the piles are spaced 6 m apart. Pile row 3 has dimensions of $2 \times 3 \times 18$ m (width × height × length), and the piles are spaced 6 m apart. These material properties are listed in Table 1.

As shown in Fig. 15a, when the slope is pre-reinforced with pile row 1 prior to initial excavation, the maximum shear strain is located along weak layer 2 after final excavation. The $F_s$ of the piled slope is calculated as 1.092. As seen in Fig. 15b and c, when pile row 1 is installed prior to initial excavation and pile row 2 is installed prior to the second stage of excavation, the maximum shear strain is located at the toe after final excavation; the $F_s$ of the piled slope is 1.203; when pile row 1 is...
Fig. 12. Progressive evolution nephograms of the maximum shear strain and the critical failure surface in basalt slope (a) no excavation (b) after first stage of excavation (c) after second stage of excavation (d) after third stage of excavation (e) after pile and anchors were installed.
installed prior to initial excavation, and pile row 3 is installed prior to the third stage of excavation, the maximum shear strain is mainly located along weak layer 2 after final excavation, and the calculated $F_S$ of the piled slope is 1.195. The critical failure surface is clearly minimized by pre-reinforced piles, and their positions and the slope stability varies with the position of pile rows. The obtained $F_S$ values for pre-reinforced slope are greater than the ones for the non-reinforced slope and also significantly exceed 1.0. According to the specifications JTGD30-2015, the safety factor of 1.05 is required for short-term slope stability; therefore, it can be reasonably considered that the pre-reinforced piles have a substantially advantageous impact on preventing the slope failure. Moreover, double rows of piles are more effective in improving the slope stability compared with a single row of piles. The slope stability exhibits little change when the position of the lower row of piles is changed. Nevertheless, the significant impact of the upper row of piles (i.e., pile row 1) on the slope stability is apparent, indicating a primary pre-condition for securing the slope safety is the upper row of piles (i.e., pile row 1); thus, the piles should be first installed in the crest of the slope before initial excavation.

10. Conclusions

The behaviour of the basalt slope with dip weak layers was investigated in this case study. Based on the results of the field investigation, FEM analysis and LEM analysis, the main conclusions are summarized as follows:

1) Roadway excavation and the two dip weak layers contributed to the basalt rock slide. The relationship between roadway excavation and weak layers is therefore the focus of significant attention in roadway engineering during both site reconnaissance and construction; in particular, roadway excavation should minimise exposing dip weak layers to prevent the potential slide.

2) Two or multiple weak layers contributed to the complex slope behaviour during the excavation progress; the position of the failure surface of the slope varies with increasing excavation, and two clearly defined failure surfaces are ultimately developed along the two dip weak layers. The slope failure is characterized by multiple translational slides. Significant attention should be placed on the evolution of the failure surface during excavation progress to correctly evaluate the slope stability.
3) Pre-reinforced piles can essentially change the slope behaviour and improve the slope stability; therefore, pre-reinforced piles with favourable pile positions should be installed prior to excavation in the slopes with dip weak layers.

Acknowledgements

This research is financially supported by the Cultivating Program of Excellent Innovation Team of Chengdu University of Technology (HY0084) and the Key Program of State Key Laboratory of Geohazard Prevention and Geoenvironment Protection (2011Z002).

References


