Block stability analysis considering depth-dependent damage on bedding planes due to blasting

Bo Lu[1,a], Dongdong Xu[1,b], Jiliang Wang[2,c*]

[1]Key Laboratory of Geotechnical Mechanics and Engineering of the Ministry of Water Resources, Yangtze River Scientific Research Institute, Wuhan, Hubei province, China
[2]Three Gorges Geotechnical Consultants Co., Ltd. (Wuhan), Wuhan, Hubei province, China

[a]88504734@qq.com, [b]xudd@mail.crsri.cn, [c*]39128518@qq.com

Abstract. The study presents a case study on the stability analysis of a rock slope using block theory. During the early stages of excavation of the intake slope, which is a dip slope with the bench face angle greater than the dip angle of the bedding planes, local failures due to sliding along bedding planes occurred. Then, the slope stability was re-evaluated using a two-dimensional (2-d) limit equilibrium method with new strength parameters of the bedding planes obtained through back-analysis. If overall sliding along bedding planes is considered, according to the re-evaluation results, the corresponding FOS will be less than 1.0 even if reinforcement is added. In view of this problem, a thorough investigation aimed at the cause of the sliding failures and the evaluation of the slope stability was performed. This study proposed consideration of the weathering unloading of shallow strata and the damage depth of blasting and reasonably utilised the lateral constraint of the downstream transverse slope sections. Based on the analytical results from block theory, the study proposed suggestions for slope support designs and measures for construction safety which have been adopted and implemented by various design organisations. The results have been verified by subsequent excavation.

Keywords: rock mechanics; block theory; slope stability; bedding plane;

1 Introduction

Since the end of the 1970s when Genhua proposed block theory [1,2], after 30 years of development and improvement, the theory has become an effective method for analysing the stability of engineering rock masses. The basic assumptions of block theory are as follows: (1) Structural planes are perfectly planar and are assumed to extend entirely through the volume of interest; (2) Rock blocks defined by the system of structural planes are assumed rigid and; (3) The instability of rock masses is shown to be governed by shear sliding along structural planes. Block theory has two basic theorems including a finiteness theorem and a removability theorem, which have been proved using mathematical methods by Genhua. In 1985, Goodman and Genhua pub-

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lished the first monograph on block theory [3], which indicates that this theory has been recognised by the international rock mechanics and engineering community. In view of the theory and its governing system, block theory is rigorous and complete, so there is no room for follow-up researchers to develop the theory further. Therefore, related research achievements in the literature mainly focus on specific applications of the theory [4-6].

When being applied to solve practical engineering problems, block theory is also faced with the same problem as numerical methods, that is, "garbage in: garbage out", namely, the effective output relies on effective input. In terms of block theory, the effective input refers to the reliability of shear strength parameters of any sliding planes. In an excavation, the mechanical properties of structural planes inevitably endure different degrees of disturbances and are strengthened, or weakened, due to differences in the types of engineering plant used, and the different angles between the occurrence of structural planes and any excavation surfaces. For instance, the steep-inclined structural planes extending along the axis direction in the top arch of an underground cavity are strengthened due to the squeezing compaction caused by the tangential stress concentration around the cavity (i.e. the normal stresses on the structural planes increase). Structural planes which occur nearly parallel to free surfaces are significantly weakened due to the excavation. Generally speaking, the structural planes with similar occurrence loci to excavated free surfaces are sensitive to excavation unloading and blasting vibration. This is the fundamental reason why sliding instability occurs due to the significant degradation of the mechanical properties of the bedding planes when excavating dip slopes. The spallation induced by blasting vibration is also a concern for many scholars [7-9]. However, it is difficult to quantify the damage caused by excavation unloading and blasting vibration on the structural planes and relevant studies are rare at present.

A hydropower station is located in a typical, deep, V-shaped valley with the close-to-river slope height exceeding 800 m. There are three diversion tunnels located on the right bank of the Jinsha River. Due to the sliding occurring along bedding planes in the early excavation of the intake slope of a diversion tunnel on the right bank of this hydropower project, the overall slope is likely to be unstable. In view of this problem, this study proposed consideration of the weathering unloading of shallow strata and the damage depth of blasting and reasonably utilised the lateral constraint of the downstream transverse slope sections. Based on the analytical results from block theory, the study proposed suggestions for slope support designs and measures for construction safety which have been adopted and implemented by various design organizations.

2 THE DILEMMAS IN SLOPE DESIGN

2.1 Basic geological conditions

The bed rock in the intake slope of the diversion tunnel on the right bank mainly is Pt_2y_1 flesh-pink interbedding with thin- and medium-thick layered quartzose marbleized dolomites, showing a uniform lithology. Structure planes are mainly found to be bedding fractures (Fig. 1). The occurrence of rock strata significantly fluctuates in space. According to the relationship between the occurrence of rock strata and slope orientation, the intake slope demonstrates three slope structures from upstream to downstream: dip slope, inclined slope (0 - 30 m to 0 + 20 m) and transverse slope. The slope rock is of IV_1 grade in the dip and inclined slope sections, while that of the transverse slope is of grade IV_2[10].

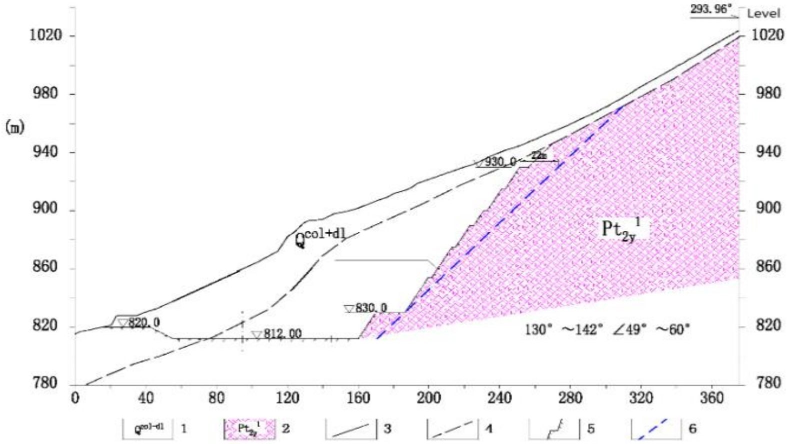


Fig. 1. Engineering geological profile and overall failure model

The dip slope was excavated at a trend of 10° on average and was inclined to the SE, showing a slope excavation ratio of 1:0.55, that is, the dip angle of the excavation slope was 61°. The rock strata exposed in this slope range were generally trended to 37° and inclined to the SE with a dip angle of 50°. The recommended values of the bedding plane parameters are listed in Table 1. According to the geological investigations undertaken in the early stage, and geological surveys carried out during construction, the bedding planes had stable occurrence and extensibility with small spacing. Therefore, they exerted significant influences on the stability of these sloping rock masses.

Table 1. Recommended values of mechanical parameters of structural planes in the intake slope of the diversion

Type	Feature	F'	c'(MPa)
Bedding plane	Filled with phyllite	0.5~0.6	0.08~0.1
Fracture (no bedding plane)	Without filling	0.7~0.8	0.1~0.2

Owing to the dip angle of bedding planes being smaller than the inclination of the slope (according to the design), local failures possibly tended to occur due to sliding along bedding planes in a single bench (Fig. 2). Overall sliding along bedding planes may also occur in the final slope formed after completing the excavation (Fig. 1). By using the parameters in Table 1, the designer evaluated the slope stability and support parameters of the slope by using the two-dimensional (2-d) limit equilibrium method before any excavation began.

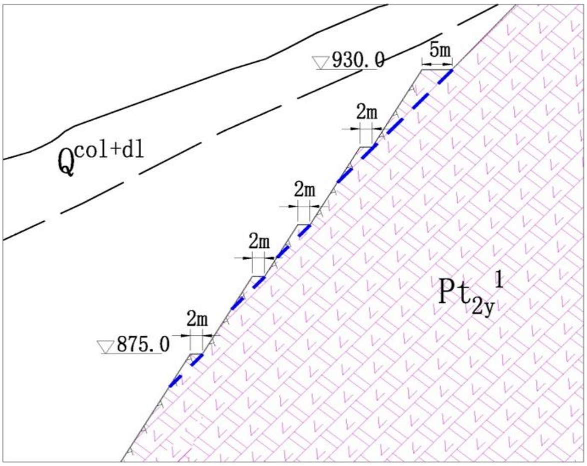


Fig. 2. Local failure modes

2.2 Local failures due to sliding along bedding planes

When the slope was excavated to the bench at an elevation of 915 m, local collapses due to sliding along bedding planes occurred (Fig. 3). By observing the failure surfaces in situ, it can be seen that the shallow surfaces were weathered with smooth surfaces and slight fluctuations. In addition, a set of unfilled rigid structural planes vertical to the slope was developed in the downstream dip slope (Fig. 4), and this set of structural planes formed the lateral cutting boundary of blocks sliding along bedding planes in the dip slope.

Fig. 3. Single sliding failure along bedding planes in the excavation

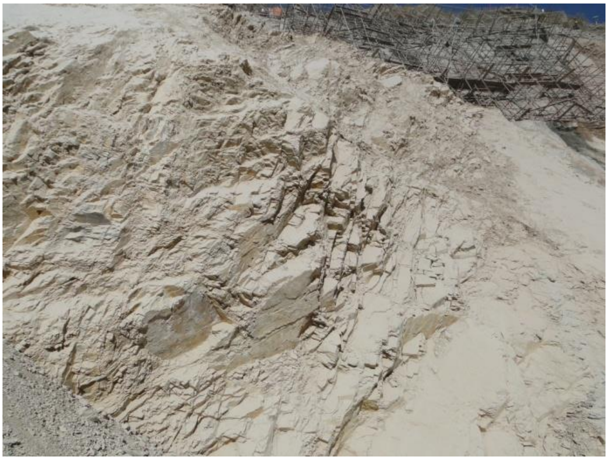


Fig. 4. Steep-inclined structural planes nearly vertical to the slope developed in the downstream dip slope

According to personnel on site, these sliding failures generally occurred two to three days after exposing the excavation surfaces and there was no obvious acceleration in the sliding process. This phenomenon suggests that: (1) The local sliding failures may be related to damage to the mechanical properties of the bedding planes caused by blasting vibration and excavation unloading, and the large decrease of shear strength of bedding planes directly caused the FoS of local blocks to approach a critical value. (2) The designed support scheme failed to be supplemented in time which was also an important reason for the occurrence of the local failures.

2.3 Analysis of local sliding and collapses

Formula (1) is proposed to calculate the safety coefficient of blocks under the conditions of single sliding based on block theory.

$$K = \frac{N_i \cdot f_i + C_i \cdot A_i}{T_i} \tag{1}$$

Where, f_i and C_i indicate the friction coefficient and the cohesion of sliding surface i, respectively; A_i represents the area of the sliding surface; N_i and T denote the components of the resultant force $\vec{R}$ along the normal $\vec{n}_i$ and the tangential (movement direction) of the sliding surface, and are expressed as:

$$N_i = \left|\vec{R} \cdot \vec{n}_i\right| \tag{2}$$

$$N_i = \left| \vec{R} \times \vec{n}_i \right| \tag{3}$$

Without considering the active supporting forces, the resultant force is the dead weight of blocks.

It can be seen from Formula (1) that N_i and T_i are directly proportional to the resultant force, namely, the dead weight of the blocks, which is directly proportional to the cube of the characteristic length l of the blocks. In addition, the area A_i of the sliding surfaces is directly proportional to the square of l, that is:

$$N_i \propto l^3 \tag{4}$$

$$T_i \propto l^3 \tag{5}$$

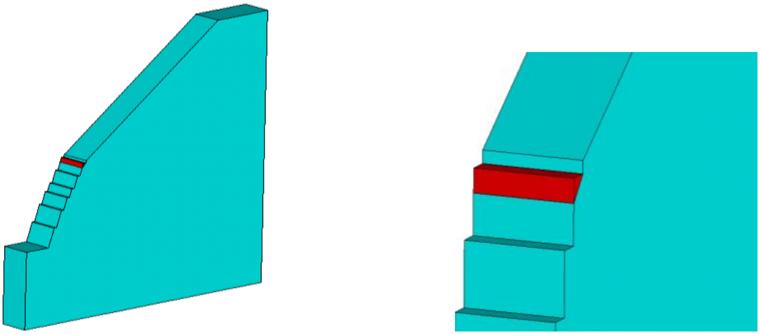
$$A_i \propto l^2 \tag{6}$$

Combining this with Formulae (1) and (4) ~ (6), it may be inferred that, in the case of similar morphology and failure mode, the larger the volume of the blocks, the lower the FoS.

Blocks experienced sliding and collapses along bedding planes on a single bench and have similar geological forms and failure modes to those subjected to overall sliding failure on the final excavated slope. For this reason, according to the inference that the larger the volume of blocks, the lower the FoS, slopes subjected to this global failure mode had a lower FoS.

The failures of blocks on the single bench at an elevation of 915 m in the excavation were closely related to the damage to the mechanical properties of the bedding planes caused by excavation unloading and blasting vibration. Considering these, the designer firstly carried out a back-analysis using the parameters of the bedding planes under such disturbances and then re-evaluated the overall stability of the slope in accordance with the new parameters.

Table 2 summarizes the results of the back-analysis based on the sliding and collapse failures along the bedding planes in the excavation of the slope between elevations of 930 and 915 m. Herein, by using the most unstable block on the single-stage slope between benches EL915 m and EL930 m as the research objects for back-analysis of parameters of subsequent bedding planes, the geometrical shape is shown in Fig 5: the block was formed by cutting the rear of the bench at EL930 m using the bedding plane with a dip angle of 50° and trended in the same direction as the slope. Owing to the block having been exposed on the surface of the slope and buried shallowly, it was affected by blasting vibration, the effects of sliding resistance of the lateral constraints were not considered, but only the dead weight of the block was considered. The calculated results for the block were acquired by combining the recommended, and the new, parameters obtained through back-analysis (Table 2).



a. The overall model of the block on the slope b. Locally magnified view of the block

Fig. 5. The most unstable block on the single-stage slope between EL915 m and EL930 m

Table 2. Results of back analysis of parameters of bedding planes

Block information	friction coefficient	cohesion(KPa)	FOS
Volume: 275.46 m^3	0.50	80.0	3.154
		90.0	3.181
		100.0	3.207
	0.55	80.0	3.515
		90.0	3.542
		100.0	3.568
	0.60	80.0	3.876
		90.0	3.903
		100.0	3.929
Area of the bottom sliding surface: 239.72 m^2	0.50	14.00	1.002
		13.00	0.961
		12.75	0.950
		12.00	0.919

In general, the influences of excavation unloading and blasting vibration on the cohesion of the bedding planes are more significant than those on the friction coefficients. Therefore, when conducting back-analysis for the parameters of the bedding planes, the friction coefficients are fixed to allow back-analysis for only the cohesion of the bedding planes. Furthermore, to investigate the effects of construction disturbance on the friction coefficients of the bedding planes, in the back-analysis, the friction coefficient values formed the lower bound of the parameter interval in the undisturbed state (Table 1). Based on the back-analysis results, when the friction coefficient values were at the low value end of their range, and the cohesion of the bedding planes fell to 14 kPa, the FoS of the exposed block on the single bench between elevations 930 m and 915 m against sliding along the bedding plane is about 1.0, that is, it was in a critical sliding state. While when the lower limits on the friction coefficient and cohesion of the bedding planes are taken, the FoS of the block is greater than 3.0. The weakening of the shear resistance of the bedding planes because of the damage to the mechanical properties of the bedding planes induced by construction disturbance is the fundamental reason behind the sliding and collapse of the block along the bedding plane in the excavation of the slope between the elevations of 930 m and 915 m.

The designers re-evaluated the overall stability of the excavated slope by using the new parameters for the bedding planes obtained from back-analysis. The results show that even if the system anchor cable (measuring 3 m × 3 m) was used to strengthen the supports, the FoS of the slope cannot reach 1.0. The existing problem is that the original parameters of the bedding planes are too high, and are inconsistent with the local sliding and collapse appearing in the excavation at an elevation of 915 m. What was worse, the parameters obtained from the back-analysis after disturbance were too low, resulting in too great a support force being required for the FoS on the global stability of the slope to meet design requirements.

3 SOLUTIONS

3.1 Existing problems

The authors believe that it is necessary to carry out back-analysis on the shear strength parameters of the bedding planes because the recommended values of geological parameters are proposed for situations without construction disturbance based on field test results in tandem with engineering analogue methods. They, thereby, fail to fully consider the influences of blasting vibration caused by field excavation on the mechanical properties of the bedding planes. However, it needs to be noted that excavation unloading and blasting vibration can only exert their influence to a certain depth, and when exceeding that depth, the bedding planes are unaffected by construction disturbance. Therefore, the parameters for the bedding planes obtained from the back-analysis according to the collapse failure of blocks can only represent the bedding planes when the shear capacity of shallow surfaces largely decreases due to the significant disturbance to the free surfaces of the slope. Thus, they cannot be directly used for re-evaluating the stability of the slope under the global failure mode. The reason is that owing to the bedding plane used for the sliding surface having been deeply buried, the shear strength parameters of the bedding plane should be greater than those obtained by back-analysis.

Furthermore, the 2-d limit equilibrium calculation did not consider the lateral sliding resistance of the lateral cutting boundary on the slope in question. In accordance with the occurrence of bedding planes and the designed trend of the slope, the bedding plane showed a 27° angle with the free surface. This suggests that the lateral sliding resistances of the adjacent slopes did exert some effect.

3.2 Several problems considered in the calculation

(1) Values of the parameters of the bedding planes

It can be shown that, upon excavation, the height of the slope will increase and the potentially largest block exposed on the excavation surface will increase in volume.

At the same time, the depth of the underlying sliding plane increases, thus the damage due to blasting decreases and even disappears.

The influences of blasting loosening effects on the bedding planes are equivalent to the decrease in cohesion. For the sake of simplification, it is considered that from the shallow-burial depth bedding planes that are affected by the most dramatic construction disturbances to the undisturbed bedding planes at a certain burial depth, the range of cohesion values increases linearly from 0 to 80 kPa. Moreover, the friction coefficient of the bedding planes takes its values as that at the lower limit of the range of recommended geological, i.e., 0.5.

(2) Sliding resistance of lateral constraints

Due to the unloading effects, the rock mass in the shallow regions of the slope is in a low-stress state. Therefore, the normal stresses on the lateral cutting boundary (Fig. 3) of the block are not considered. To take the effects of blasting loosening and influences of weathering unloading of rock masses, the value of the cohesion on the lateral cutting boundary increases linearly from 0 to 200 kPa from the shallow surface to the interior of the slope.

(3) Disturbance depth

Fig. 6 shows the curves from sonic wave tests on two typical sections of the dip slope: the depth of excavation unloading of the dip slope is generally less than 15 m. To analyse the stability of blocks, the depth was taken as a uniform 15 m for the sake of convenience.

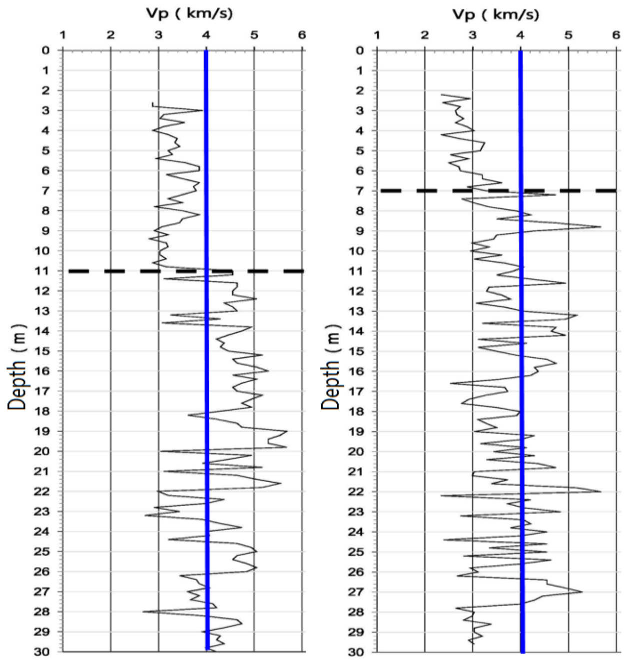
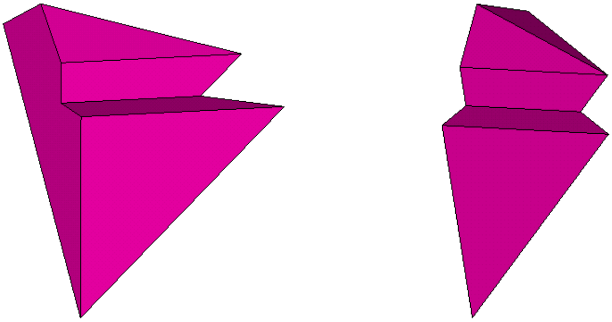


Fig. 6. Sonic wave test results: the dip slope

4 Calculation results

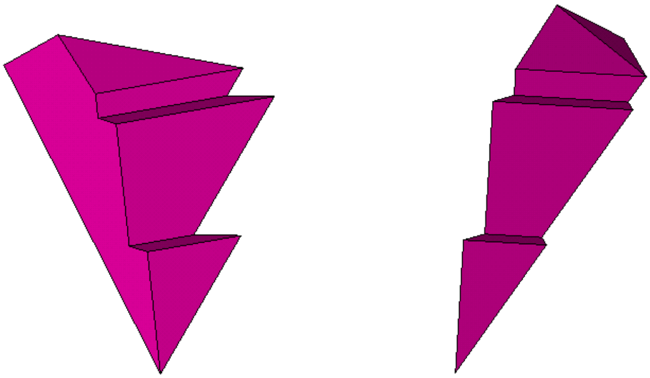
Fig.7 to Fig.12 show the maximum volume of blocks possibly exposed and sliding along bedding planes on each bench in the excavation of the slope respectively. The reason for selecting the blocks with the largest volumes for stability re-evaluation is presented in Section 2.3. Table 3 lists the analysis results relating to block stability: the "Required anchor cable" in the last column of Table 3 refers to the number of anchor cables required to make the FoS of corresponding blocks no less than 1.25, when only the effects of the anchor cables are considered. The load borne by each anchor cable is designed to be 200 t.

Fig. 13 shows the maximum horizontal burial-depth of the potentially largest blocks at each elevation of the slope: from the bottom EL830 m bench to the upper EL930 m bench, the maximum horizontal burial depths of these blocks are 15.40 m, 25.88 m, 32.50 m, 35.92 m, 42.55 m, and 24.73 m, respectively. These depths are an important reference for designing anchor cables.



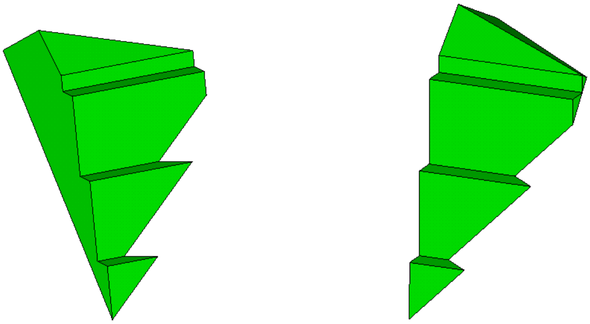
a. Left view b. Right view

Fig. 7. The largest block possibly appearing in the excavation of the slope at EL915 m



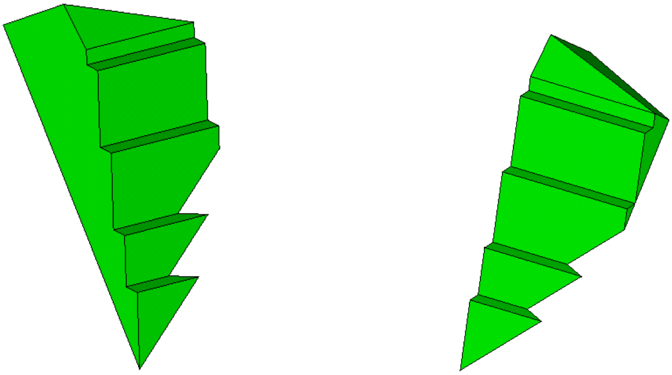
a. Left view b. Right view

Fig. 8. The largest block appearing while excavating to EL900 m



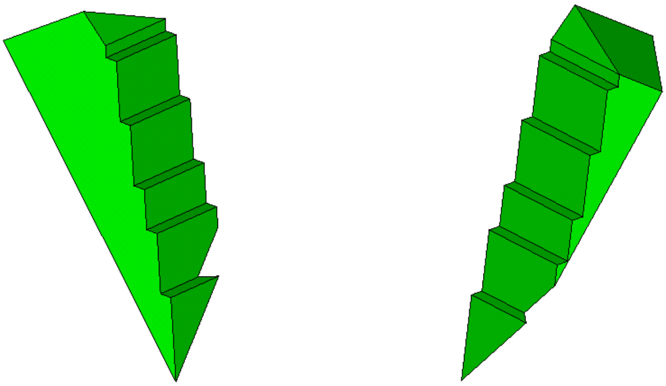
a. Left view b. Right view

Fig. 9. The largest block probably occurring in the excavation of the slope to EL890 m



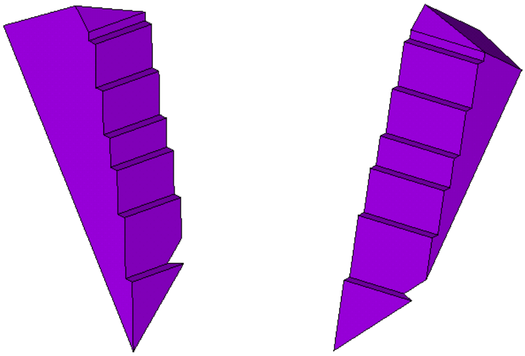
a. Left view b. Right view

Fig. 10. The largest block likely to appear when excavating to EL875 m



a. Left view b. Right view

Fig. 11. The largest block possibly appearing when excavating to EL854 m



a. Left view b. Right view

Fig. 12. The largest block probably occurring when excavating to EL875 m

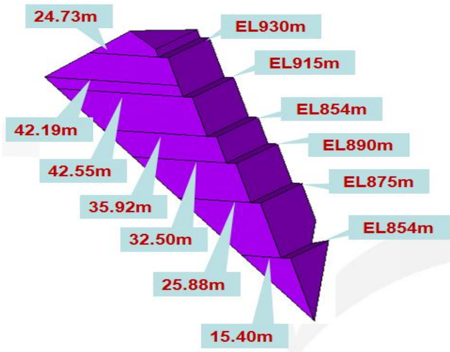


Fig. 13. Reinforcement depths of the underlying largest blocks on the slope

Table 3. Analysis results for the stability of the potentially largest blocks on each bench

Excavation elevation (m)	FOS (without support)	Required anchor cables
915	0.789	4
900	0.932	10
890	1.096	12
875	1.144	14
854	1.153	29
830	1.199	29

Based on the calculation results listed in Table 3, when approximately considering the effects of blasting loosening and weathering unloading of shallow rock masses by weakening of their cohesion, the parameters of the bedding planes and the sliding resistances of the lateral structural planes gradually increase from the surfaces of the slope to the interior thereof. As the slope was excavated continuously, the FoS on the stability of the largest movable block possibly appearing on the free surfaces of the slope rises gradually. When the slope was excavated to the bench at EL890 m, on account of the shear capacity of the bedding planes and sliding resistances produced by rigid lateral structural planes, the largest block that may appear on the excavated surfaces of the slope can be self-stabilized; however, the FoS of the largest blocks that may occur above the bench at EL890 m are less than 1.0.

In fact, while excavating to 910 m, even if the supports were not implemented timeously, local failures of the blocks that span two adjacent benches, as shown in Fig. 8, did not take place, indicating that the calculation parameters used were conservative.

5 ENGINEERING VERIFICATION

From the above analysis, with increasing depth, the parameters of the bedding planes and rigid structural planes that play a role in the lateral cutting show that the planes change from a state of complete disturbance to an undisturbed state with gradually enhanced mechanical properties. With the gradual strengthening of the shear capacity, the FoS of potentially unstable blocks on each level of benches which are likely to slide along the bedding planes gradually increases. Therefore, the stability and safety of the slope are guaranteed in the excavation [11].

Based on the results of the 3-d stability analysis of the blocks, the designer urged the contractors to take two important measures including strictly controlling blasting operations and constructing supports timeously, which effectively ensured the safe excavation of the slope. The subsequent excavation verifies that the stability calculations are reliable. Fig. 14 shows the displacement-duration curves at a typical multi-point displacement meter, which show that the deformation is basically convergent.

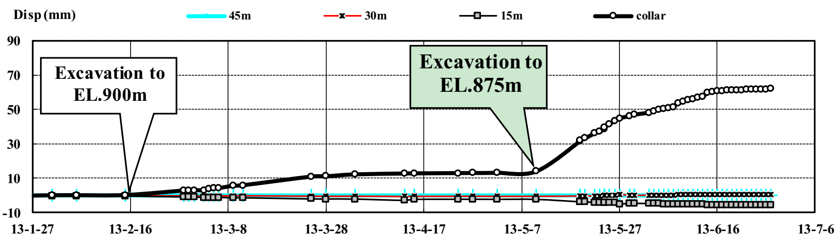


Fig. 14. Deformation-time curves monitored in the typical section of the dip slope

6 Conclusions

A thorough investigation aimed at the cause of the sliding failures and the evaluation of the slope stability was performed. Apparently, the local failures were caused by blasting and a lack of timely support, which resulted in damage to the bedding planes and a loss of shear strength. During the re-evaluation performed by designer organisation, two important facts were not taken into account: firstly, the extent of damage to the bedding planes caused by blasting and excavation unloading would vary with depth; secondly, 2-d calculations were over-simplified, lateral constraint effects from adjacent slope sections (downstream) could not be ignored. With the two important factors mentioned above being properly considered, the calculations show that overall stability of the cut slope is out of question under existing support design conditions. Local failures could be avoided by controlled blasting and timely support works. The results have been verified by subsequent excavation.

According to the case study formed by the engineering slope investigated in this research, it can be seen that, in addition to suitable analysis tools, careful analysis and judgment for basic input conditions are also required to realize a reliable engineering calculation-based analysis.

Due to the lack of sufficient experimental data reported in existing studies, this study undertook an approximation to the parameters reflecting the damage to the bedding planes caused by excavation disturbance and assessed its behaviour at different burial depths. The research needs to be based on certain field tests, which forms the basis for future research.

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