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ORIGINAL PAPER

Weak Plane Failure of Phyllitic Sandstone: Back Analysis for Slope Stabilization and the Use of Probabilistic Approach for Design Optimization

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Abstract The main access road of Mae Mao dam hydropower project, Chiang Mai province, Thailand, has failed during a sunny day. The question has been raised of what might have triggered the failure. At the day of failure, the precipitation amount was very small and the accumulated rainfall amount in the past days was also not significant to trigger the failure. Detail geotechnical investigations were carried out including the geological survey, seismic refraction survey, bore holes drilling and various laboratory testing. The rock found at site was Phyllitic and Quartzitic sandstone. It appears that the engineering properties of Phyllitic sandstone are more susceptible for slope failure. Rock and soil slope stability analysis have been done, however none of them was found to be the cause of failure. After carefully reexamining the geologic structure, it was found that there is a weak plane layer that resulted in the failure. A suitable stabilization method has been selected as the economical remedy of the slope failure. In this case, flattening the back slope is the economic method along with the drainage system specially designed to reduce the pore water pressure of the weak plane layer. Rock bolts and rock anchors were found to be uneconomical and unsuitable. Probabilistic approach was used to optimize the engineering safety and construction cost.

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Keywords Slope stabilization · Rock slope · Cut slope · Weak layer

1 Introduction

Mae Mao hydropower project is located at Fang district, Chiang Mai province. The general geology consists of Phyllitic sandstone and Quartzitic sandstone. After 24 years of construction (since 1986), the failure of the main access road to the dam has occurred (Figs. 1, 2). The factor causing the failure is unclear. Rainfall and seismic activity are the two possibilities suspected for the failure. However, the precipitation amount was just 2 mm at the day of failure and the accumulated rainfall amount for 4 days prior to the failure was just 59.2 mm (Fig. 3). The precipitation threshold for landslide in Thailand is much larger than the records. Another possibility might be credited to the seismic activities around the site which was quite active during the past service period of the dam (Fig. 4). The site is located in 2B seismic hazard zone based on UBC. With the evidence of the seismic activities in the area, the seismic slope displacement

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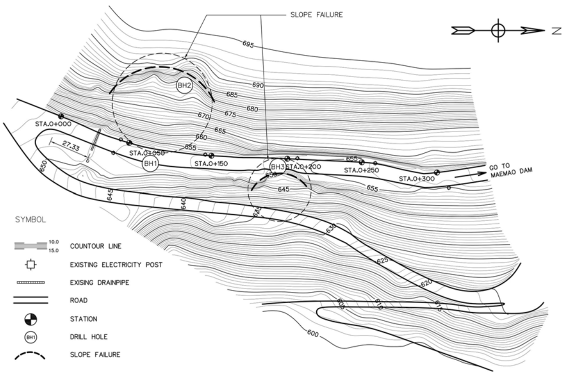

Fig. 1 Topographic elevation of the failure area

Fig. 2 Failure of the back and filled slope

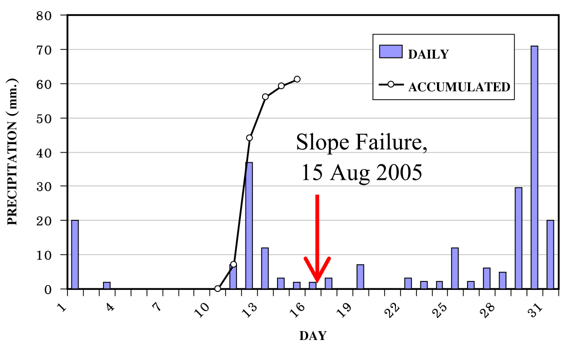

Fig. 3 Daily rainfall record in August 2005. (Mae Mao Hydropower Project 2006)

could have been accumulated for several years to eventually cause the slope failure. However, this paper is not intended to investigate the exact cause of the failure and is just limited for the safe reparation of the failed slope.

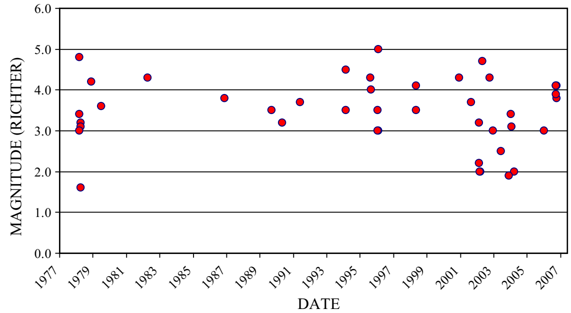

Fig. 4 Seismic record (Magnitude) from 1978 to 2007, epicenter recorded within 100 km radius from site. (Thai Meteorological Department 2007)

2 Geologic Structures

From the geological investigation, in the failure area it was found that the 15–35 m thick of Phyllitic sandstone is overlaying the Quartzitic sandstone. Figure 5 is the plan view showing the contact boundary between these two rocks. Combining with the boring log data, the schematic of geologic structure is shown in Fig. 6. The rock structures consist of fault, shear and foliation. It's the foliation plane that is firstly suspected to be the plane of failure. The orientation of this foliation plane consists of strike and dip angle of 330/30. Figures 7 and 8 show the photographs of fractures and planes. Figure 9 shows the fracture orientation data analysis on the stereonet (Markland 1972). It is found that the possible mode of rock failure is the combination of planar and wedge failure.

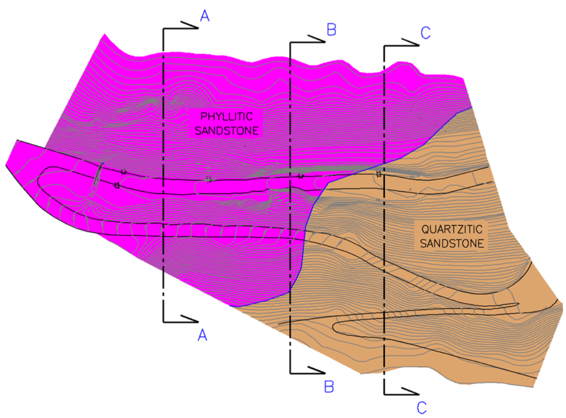

Fig. 5 Plan view showing the contact boundary between Phyllitic and Quarzitic sandstone

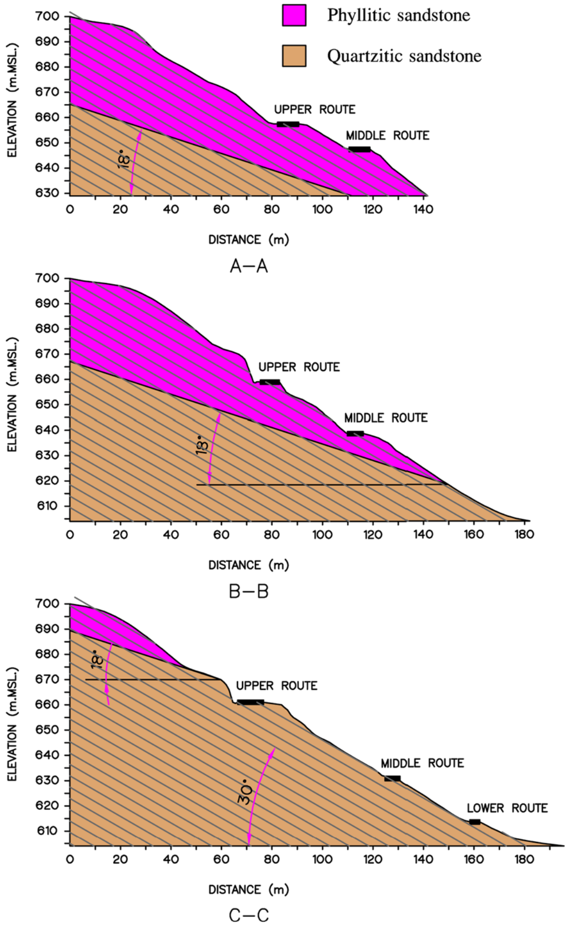

Fig. 6 Cross sections and geologic structure

Fig. 7 Rock fractures

Fig. 8 Fracture planes

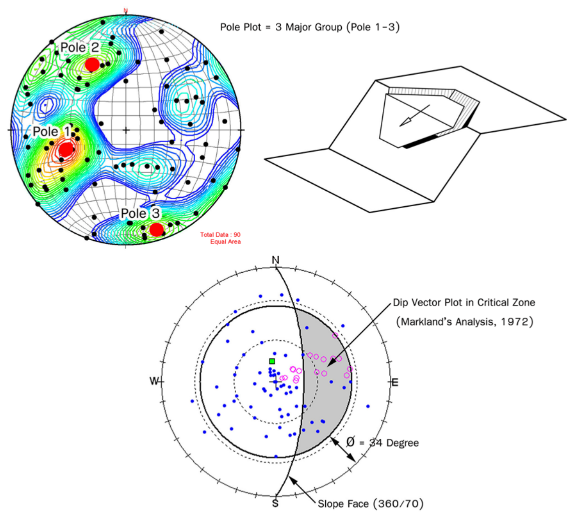

Fig. 9 Fracture orientation data analysis on the stereonet

3 Site Investigations

Site investigations were employed in order to determine the sub-surface configuration picture of rock strata, shear strength and permeability values of rock. The following activities have been carried out:

a. RMR (Bieniawski 1973) and SMR (Romana 1985) determination of the rock mass was done in the area vicinity to the failure. The results are shown in Figs. 10 and 11. It was found that the Phyllitic sandstone is mostly classified as poor to bad rock while the Quartzitic sandstone is classified as normal to good and fair rock.
b. Seismic refraction survey (Hagiwara and Omoto 1939) was done in order to investigate the thickness of weak strata. The result from seismic survey line 3 (lower slope) shows that the rock mass in this area is more solid compared to that of line 1 located in the upper slope (Figs. 12, 13).

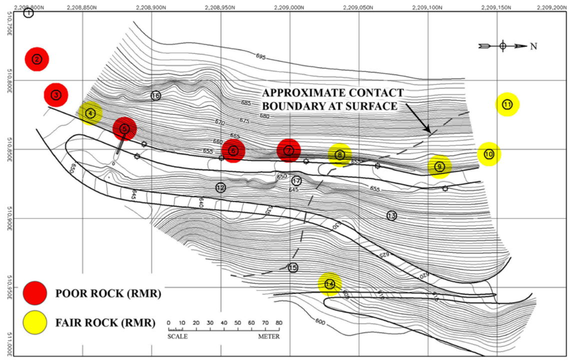

Fig. 10 Rock mass quality determination by RMR

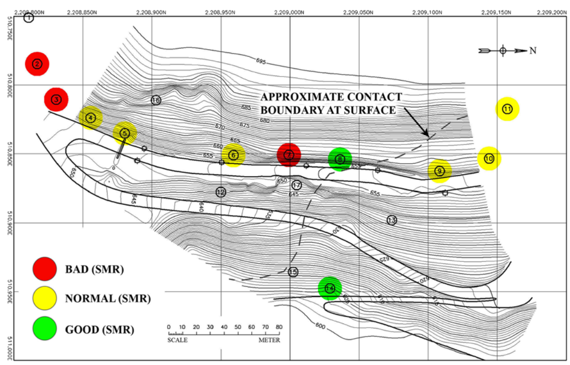

Fig. 11 Slope mass quality by SMR

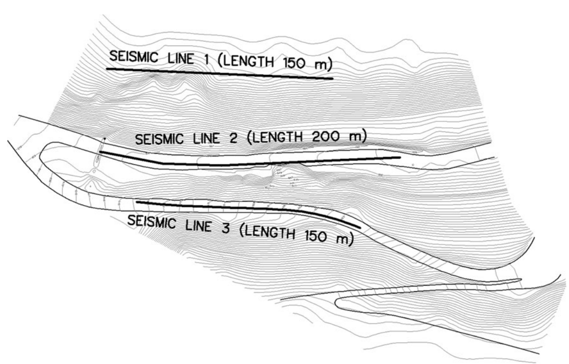

Fig. 12 Seismic refraction survey lines

This information is very useful in making the decision on repairing method.

c. Three boreholes by NX-size rock coring were done to the depth about 10–20 m. One borehole (BH-1) was drilled at the right angle to the suspected major sliding plane (dip angle of 30 degree). Part of the results is shown in Figs. 14 and 15. The results from three boreholes reveal

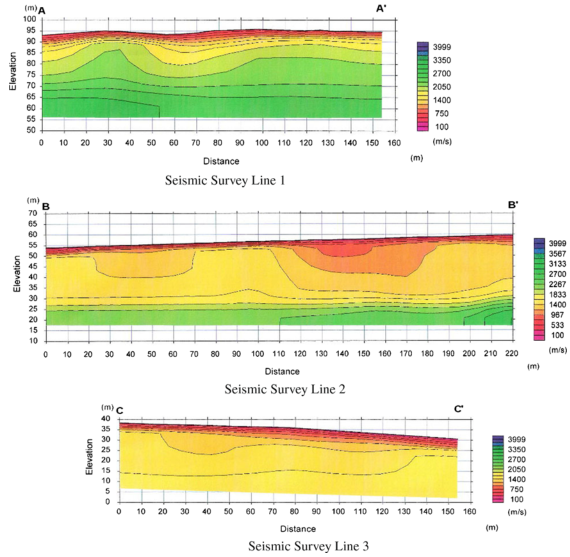

Fig. 13 Results of seismic refraction survey

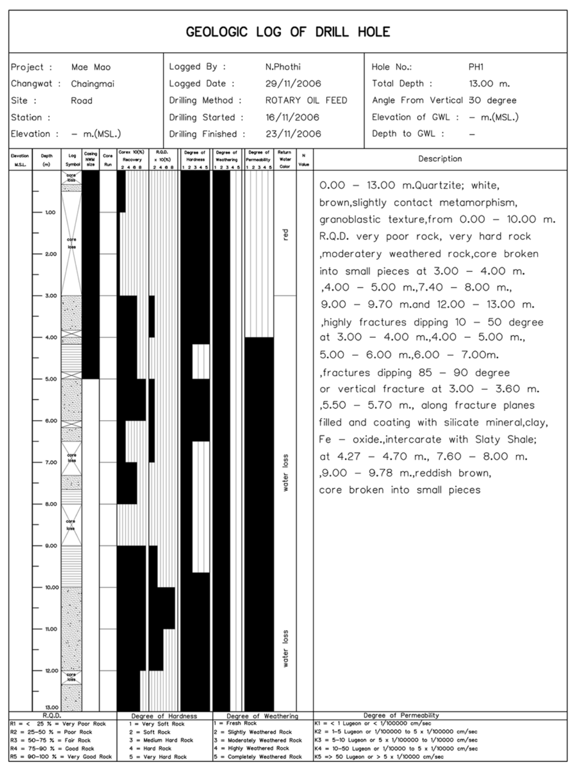

Fig. 14 Core logging of BH-1

that there is a 3 m thick completely decomposed rock layer present in each bore hole column (Fig. 16). This weak layer has a dip angle that is quite similar to the dip angle of the foliation plane

Fig. 15 Core samples showing the weak zone

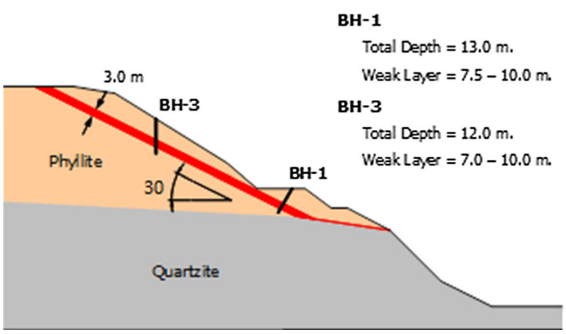

Fig. 16 Configuration and the properties of the different features of the slope for back analysis (Section A–A)

described earlier. Decomposed material in the weak layer was collected and recompacted to the estimated undisturbed state density and water content to prepare the reconstitute sample for direct shear test. Slow shearing rate was used to obtain the effective strength parameters of the weak layer material. The joint strength of Phyllitic sandstone mass was tested using the direct shear machine to obtain shear strength parameters of the natural joint and saw cut joint (ASTM 2002.). The total of 9 samples were performed and the results are shown in Table 1 and Fig. 17.

d. Rock mass permeability was determined using packer test (Hoek and Bray 1981) in order to obtain information for designing a scope for drainage design. The tests were specifically done in Phyllitic sandstone layer and the results are shown in Table 2. The results show that this rock mass is quite permeable.

4 Design Concept

Regarding the investigation results, the design concepts are comprehensively combined as follows:

a. Rock nailing is recommended instead of rock anchoring since highly weathered and fractured rock was found in which the tension in the anchored bar might not be achieved during the installation and the stressing effect might be limited in the area close to the anchor plate.
b. The concept of increasing the resisting force by adding counter weight berm at the toe of each cut slope is omitted in order to prevent the progressive failure since the rock mass in the lower portion of cut slope was found to be in higher weathering degree. This is based on the results of seismic refraction and visual geological survey.
c. The sub-surface water drainage of the slope is extensively applied in order to reduce the pore water pressure near the weak plane layer.

5 Stability Analysis and Remedial Design

Stability analysis based on the limit equilibrium method was performed for the back analysis and the analysis for slope design. KU slope computer program (Mairaing and Isaroranit 2003) was used.

a. Back analysis was done using the configuration as shown in Fig. 18. The 5 m thick layer of Phyllitic sandstone is modeled together with the 0.5 m thick of weak layer in which all layers have 30 degree of dip angle. The results of direct shear test explained earlier were used. The Phyllitic sandstone strength parameters are modeled based on its joint strength. The weak layer shear strength is modeled based on the direct shear test result of recompacted material. The result revealed that the thickness of the rock mass layer need to be at least 10 meters thick to make the factor of safety of the slope less than one (Fig. 19). This back analysis scheme is based on the assumption that the ground

Table 1 Rock properties from laboratory tests

Test property	Type of rock	
	Phyllitic sandstone	Quartzitic sandstone
Absorption (%)	1.45–6.20	0.85–2.91
Porosity (%)	3.74–14.05	2.21–7.26
γ_{sat} (t/m^3)	2.41–2.63	2.57–2.64
Slake durability	High–very high	Low–moderate
Uniaxial compressive strength (MPa)	16.45–173.07	111.60–301.31
Cohesion (t/m^2) (Joint)	1.0–4.5	–
Friction angle (degrees) (joint)	17–34	–

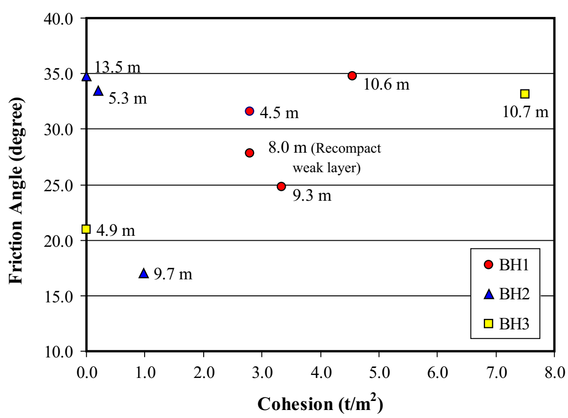

Fig. 17 Result of direct shear test

Table 2 Results of field permeability test

Drilling no.	Permeability (Lu)	RQD. (%)
BH1	300–650	0–30
BH2	90–240	0–50
BH3	45–135	0–60

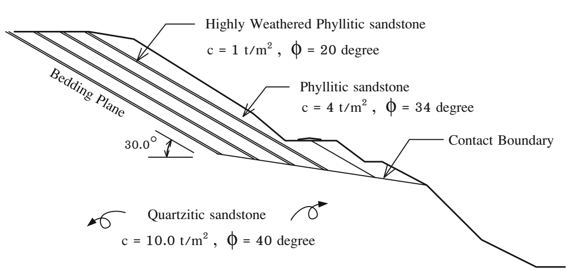

Fig. 18 Configuration and properties of slope for back analysis case (Section A–A)

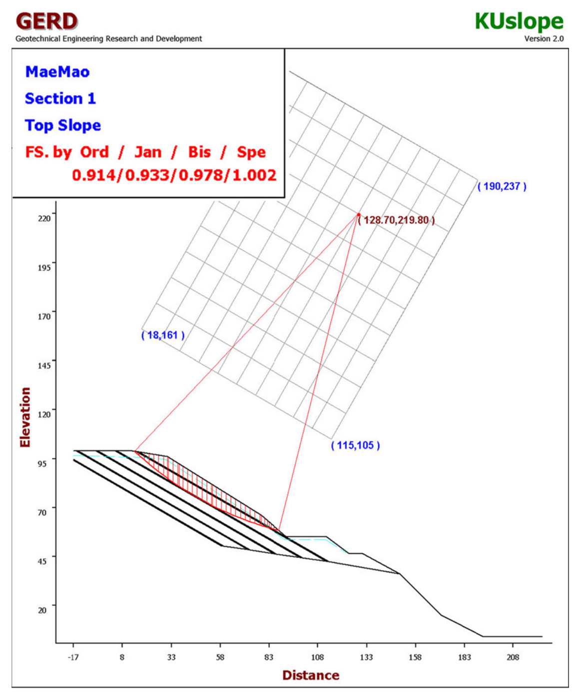

Fig. 19 Result of slope stability analysis

water is located at the ground surface. The result of the back analysis corresponds with the actual condition where the weak layer is found at the depth of 10 meters with almost 30 degree of dip angle.

Probabilistic approach was also used to calculate probability of failure (P_f) for each potential slip plane. The Monte Carlo simulation was used for determining the uncertainty of the output parameter value. “Table 3” shows the expected values and the uncertainties of the input parameters. The input and output values were assumed to be normally distributed. The uncertainties of the

Table 3 Expected values of input parameters and their uncertainties

Material	c (kN/m^2)	COV (c) (%)	SD (c) (KkN/m^2)	ϕ (Deg.)	(COV) (ϕ) (%)	SD (ϕ) (Deg.)	γ (kN/m^3)	COV (γ) (%)	SD (γ) (kN/m^3)
1	50	20	10	40	14	5.6	26	1	0.26
2	10	20	2	20	14	2.8	22	1	0.22
3	50	20	10	40	14	5.6	26	1	0.26
4	10	20	2	20	14	2.8	22	1	0.22
5	50	20	10	40	14	5.6	26	1	0.26
6	20	20	4	30	14	4.2	22	1	0.22
7	50	20	10	40	14	5.6	26	1	0.26
8	20	20	4	30	14	4.2	22	1	0.22
9	50	20	10	40	14	5.	26	1	0.26
10	20	20	4	30	14	4.2	22	1	0.22
11	50	20	10	40	14	5.6	26	1	0.26

COV (%) from Phoon et al. 1995

input parameters were expressed using the Coefficient of Variation (COV) values published by various researchers. Figure 20 shows the results of the probability of failure values for each potential slip plane. It is clear from the figure that the actual slip plane matches with the plane that has the highest probability of failure.

b. Analysis for slope remedial design was done based on the design concept explained earlier. It is recommended to flatten the upper portion of the slope including 5 m wide benching steps, 10–12 m long rock nails and horizontal drains as shown in Fig. 21. The design cases are shown in Table 4.

The ground water level is assumed to be at the ground surface and the horizontal seismic coefficient is used as 0.05. The target factor of safety of 1.5 is

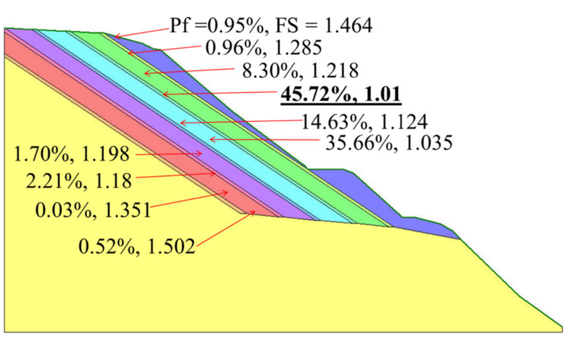

Fig. 20 Pof values for each potential slip surface

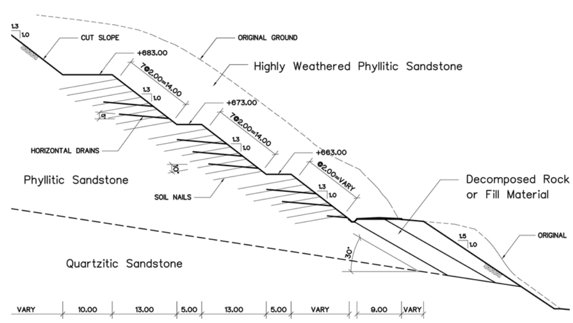

Fig. 21 Configuration of remedial work (Section A–A)

used. The results of all design cases are shown in Table 5.

As for the comparison of the construction cost, it was found that the case of solely using the cut slope of 1:1.8 yield the lowest construction cost with the favorable factor of safety (Table 6).

Therefore, this scheme was selected for the remedial work. It can also be seen from Tables 5 and 6 that use of rock nails is not a quite cost effective solution since the gaining of factor of safety is small compared to the additional cost that will have to be invested. Figures 22 and 23 shows the typical section, and drainage system surface protection measures.

Table 4 Remedial cases

Option no.	Slope (Ver:Hor)	Rock nail (W × H, m)	Horizontal drain (W × H, m)
1	1:1.3	2.0 × 2.0	5.0 × 4.0
2	1:1.5	2.5 × 2.5	5.0 × 4.0
3	1:1.8	–	5.0 × 4.0

Table 5 Stability analysis results

Option no.	Method				Seismic coefficient	Factor of safety	
	Cut slope		Rock nail			Benching	Benching + rock nail
	Slope	Cutting volume (m^3)	Dimension (m × m)	Row			
1	1:1.3	147,575	2.0 × 2.0	24	0.05	1.20	1.50
2	1:1.5	197,020	2.5 × 2.5	14	0.05	1.44	1.50
3	1:1.8	245,150	–	–	0.05	1.73	–

Table 6 Comparison of construction cost

Option no.	Construction cost (Million Thai Baht)						
	Cut slope	Rock nail	Drainage system	Erosion control	Miscellaneous works	Overhead and profit	Total cost
1	11.7	25.0	3.9	1.7	0.7	9.9	52.9
2	15.2	10.4	3.4	2.0	0.7	7.3	39.0
3	18.6	0.0	3.9	2.1	0.7	5.8	31.1

Remark Cut slope = 82 baht/m^3 Rock nail = 1000 baht/m
Horizontal drain = 1071 baht/m Erosion control = 80 baht/m^2

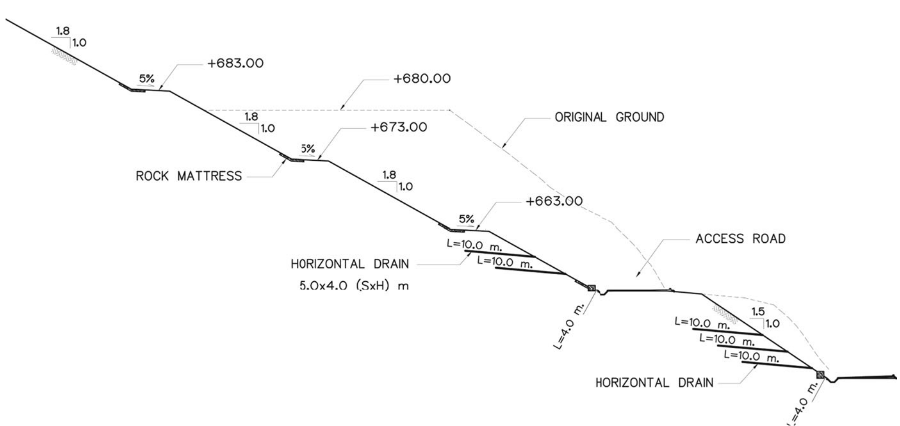

Fig. 22 Typical section of repair work (Section A-A)

6 Probabilistic Approach for Remedial Decision

Probabilistic approach has been used to obtain the final shape of the cut slope to achieve engineering safety and cost optimization. The relationship between Probability of Failure (Pf, %) and Factor of safety (FS) has been plotted in Fig. 24.

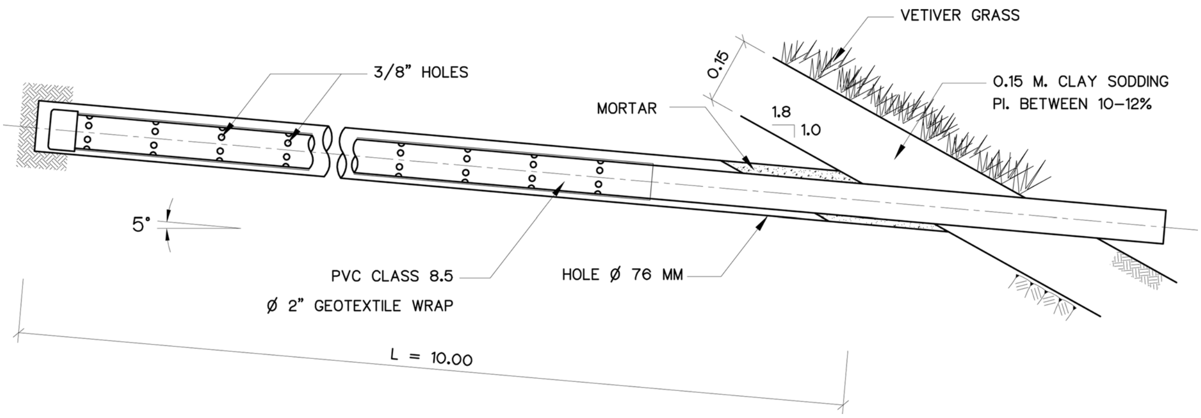

Fig. 23 Typical section of drainage and erosion control

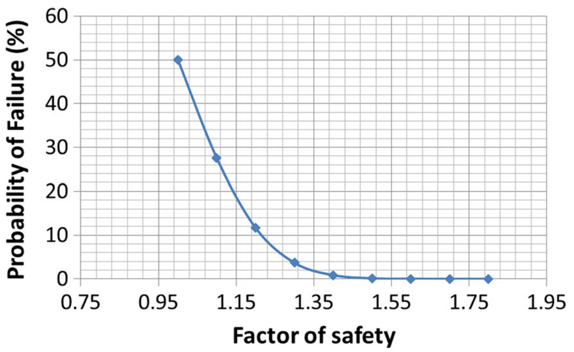

Fig. 24 Factor of safety versus probability of failure

Minimum design factor of 1.25 may be used for routing Highway (FHWA 2006). Furthermore, Santamarina et al. (1992) suggested that P_f between 1 and 2% may be used for existing large cut to low consequences of slope failure. Hence, a cut slope with factor of safety 1.3 and probability of failure of 3.69% was used for the stabilization of this slope. In order to understand the situation correctly, the results obtained were plotted in the curve proposed by Silva et al. (2008) and hence the project can be classified between category II and III (Fig. 25). The design of the project was carried out in category III considering the quality and efficiency of the contractor. The cut slope during its construction and after its completion is shown in Fig. 26.

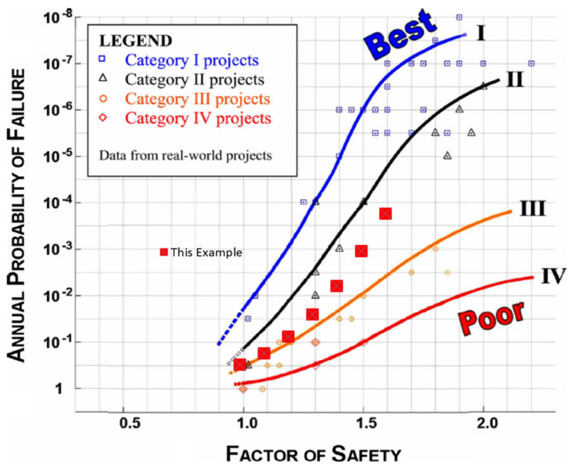

Fig. 25 Pf obtained from real world projects. Adapted from Silva et al. (2008)

7 Conclusion

The access road to Mae Mao Hydropower Project might have failed due to a weak layer found within the Phyllitic Sandstone. The actual failure condition coincides with the back analysis of slope stability. It is important to emphasize the necessity of the comprehensive geological and geotechnical survey before making the decision of the repair work. It has been found that the use of rock nails is not cost effective in repair work and hence simple cut and bench method is recommended.

Probabilistic analysis has also been used to provide additional information to the proposed slope. It was used as a tool to optimize between engineering safety and investment cost. The P_f value matches well with

Fig. 26 Cut slope during construction and after its completion

the back analysis result in deterministic analysis, confirming the possible failure plane. The relationship between Pf and FS values of repair work in this project satisfied the risk level, which can be accepted based on the service of this road.

The probabilistic approach in this project shall promote its use in the practical field and indirectly advocate extensive investigation in order to understand the uncertainty of geological and geotechnical properties of slope.

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