

Weak Plane Failure of Phyllitic Sandstone: Slope Stabilization of the Access Road of Mae Mao Dam

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ABSTRACT: The main access road of Mae Mao dam hydropower project has failed during the sunny day. The question has been arisen of what might have been triggered the failure since at the day of failure the precipitation amount is very small and the accumulated rainfall amount in the past days was also not significant to trigger the failure. Detail geotechnical investigations were done including the geological survey, seismic refraction test, bore holes drilling and various laboratory testing. The rock found at site is 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 there was none to be found of the cause of the failure. After carefully reexamine the geologic structure, it was found that there is a weak plane layer that cause the failure. Suitable stabilization method have been selected to economical remedial the slope. Simply flattening the back slope is the economical method in this case together with the drainages system that specially designed to reduce the pore water pressure of the weak layer. The rock bolt and rock anchored are found to be not economical and not suitable.

1 INTRODUCTION

Mae Mao hydropower project located at Fang district, Chiangmai 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 (Figures 1 and 2). The factor causing the failure is unclear, two possibilities have been suspected rainfall and seismic. At the day of failure the precipitation record was just 2mm and the 4 days accumulated record prior to the failure was just 59.2mm (Figure 3). Regarding to the precipitation records that causing the landslide in the country, the precipitation recorded found seems to be not enough to cause the failure. Another possibility might be credited to the seismic activities around the site which quite active during the past service period of the dam (Figure 4). With the evident of the seismic activities in the area, the seismic deformation occur in the slope could be accumulated to eventually causing the slope failure. Anyhow, this paper is not intended to investigate the exact cause of the failure. The investigation is limited just enough for safely repair the failed slope.

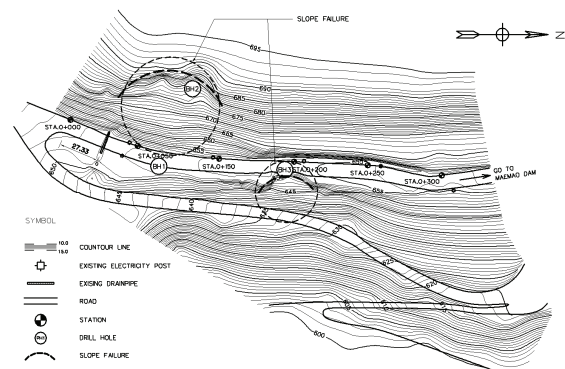


Figure 1: Topographic of the failure area.



Figure 2: Failure of the access road.

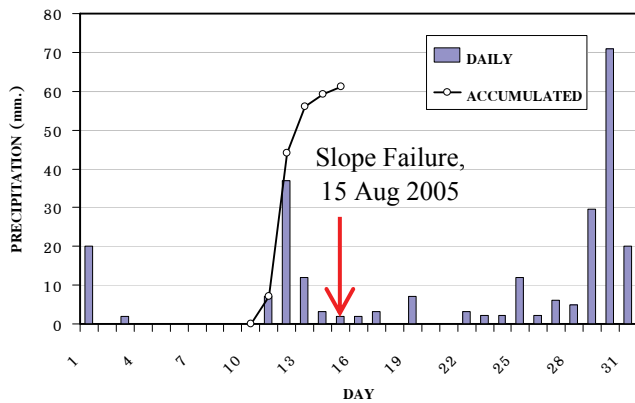


Figure 3: Daily rainfall record in August 2005 (Mae Mao Hydropower Project, 2006).

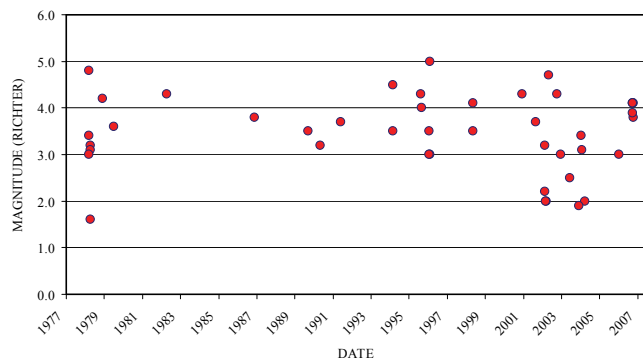


Figure 4: Seismic record (Magnitude) from 1978 to 2007 (The Meteorological Department, 2006).

2 GEOLOGIC STRUCTURES

Geological investigation was done and found that the 15-35 meter thick of Phyllitic sandstone is overlain the Quartzitic sandstone. Figure 5 is the plan view show the rock contact boundary. Combining with the boring log data, the schematic of geologic structure can be shown in Figure 6. The rock structures consist of fault, shear and foliation. It's the foliation plane that 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 picture of fractures and planes. Figure 9 shows the analysis of Stereonet (Markland, 1972).

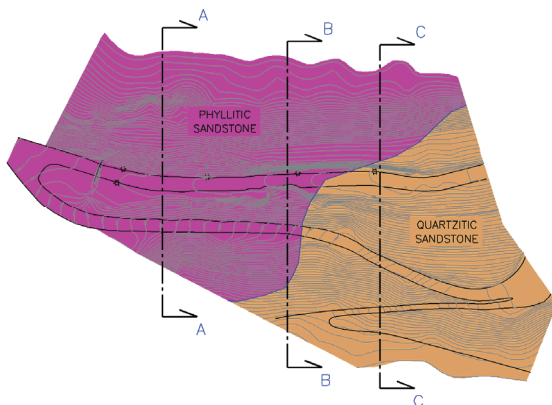


Figure 5: Plan view showing the contact zone.

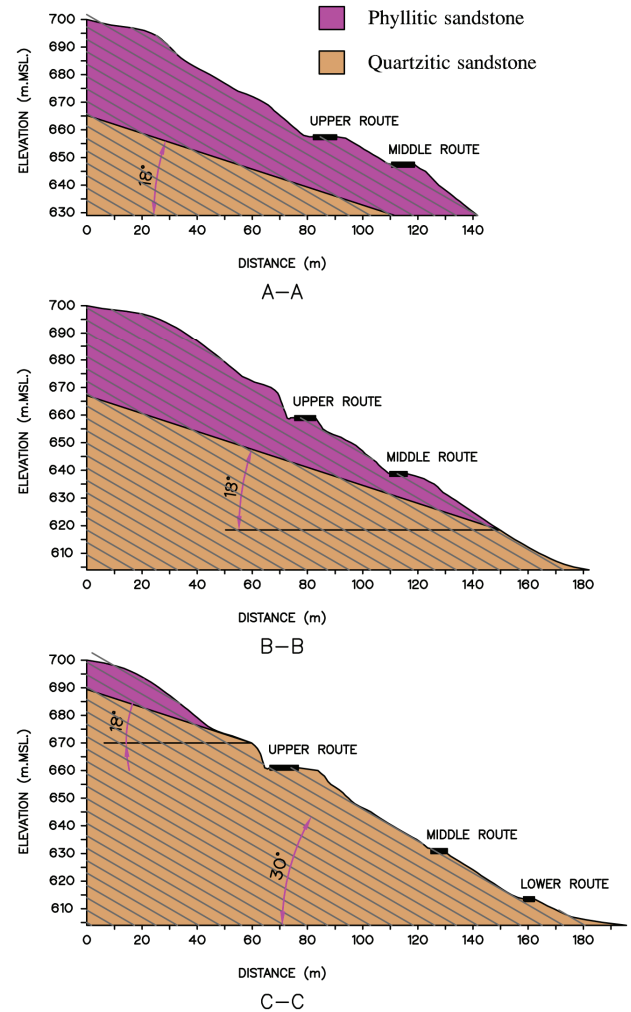


Figure 6: Cross sections and geologic structure.

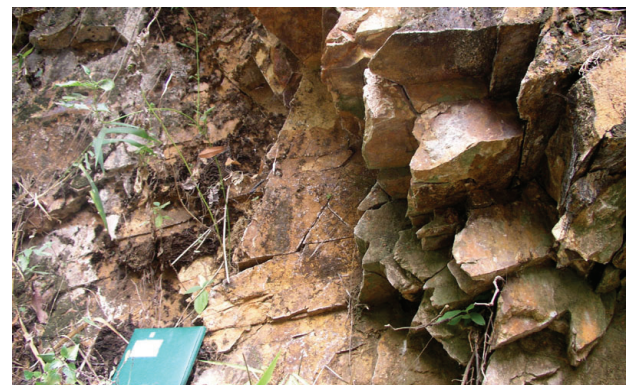


Figure 7: Rock fractures.



Figure 8: Fracture planes.

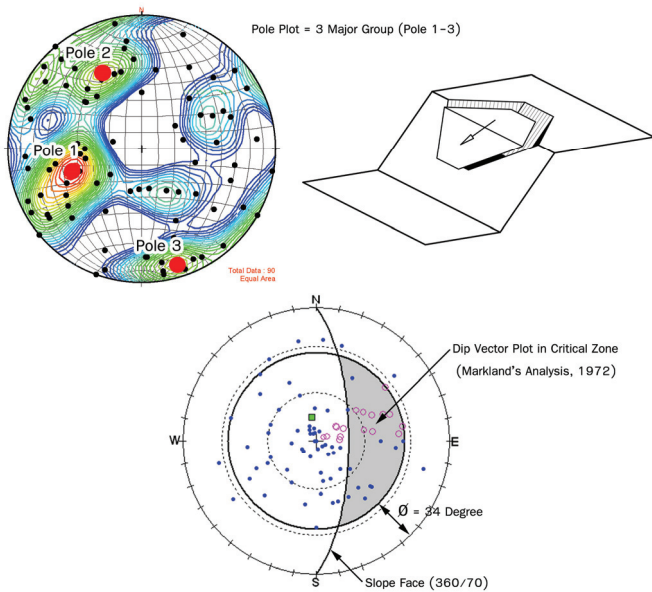


Figure 9: Stereonet analysis.

3 GEOTECHNICAL INVESTIGATIONS

Geotechnical investigations were employed in order to obtain the underground picture of rock strata, shear strength and permeability of rock. The followings have been done.

3.1 RMR (Bieniawski, 1973) and SMR (Romana, 1985) determination of the rock mass in the area vicinity to the failure were done . The results are shown in Fig 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.

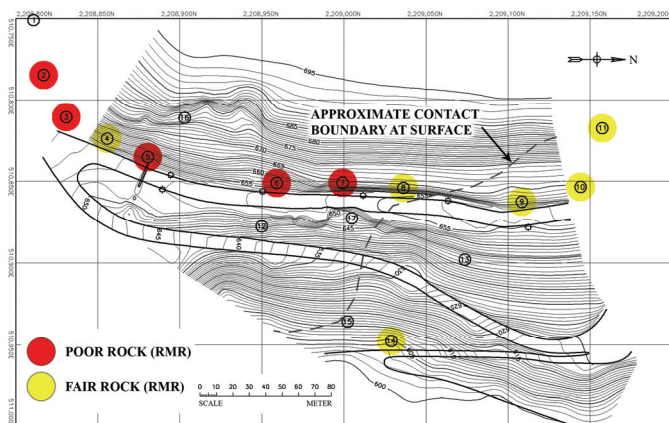


Figure 10: Rock mass quality determination by RMR.

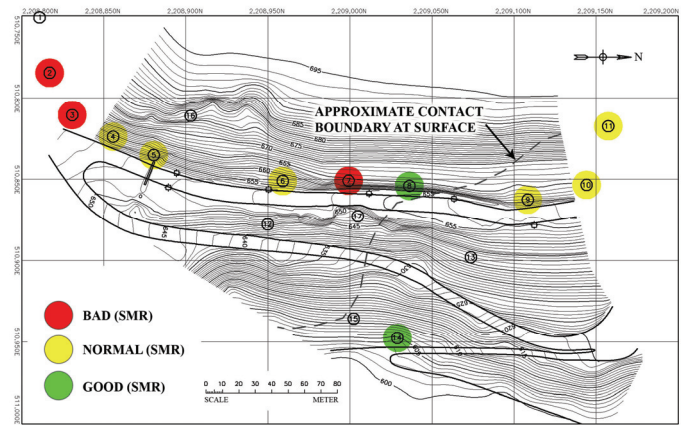


Figure 11: Slope mass quality by SMR.

3.2 Seismic refraction survey (Hagiwara and Omoto ,1939) was done in order to investigate the thickness of weak strata. The results found that, at the lower portion of the hill (line 3), rock mass presents less density than the upper portion (line 1) (Figures 12 and 13). This information is very useful in making the decision of repairing method.

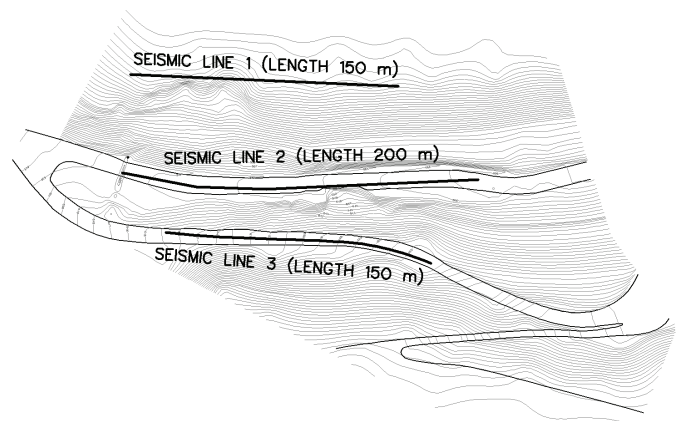


Figure 12: Seismic refraction surveying lines.

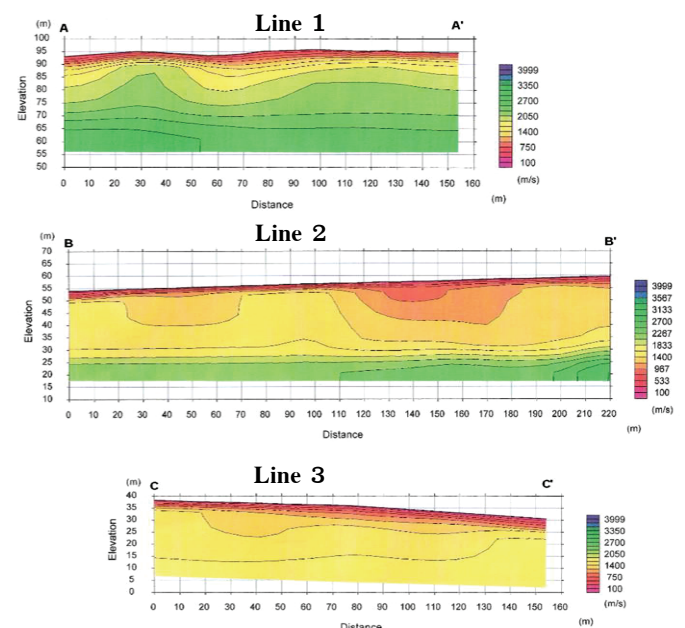
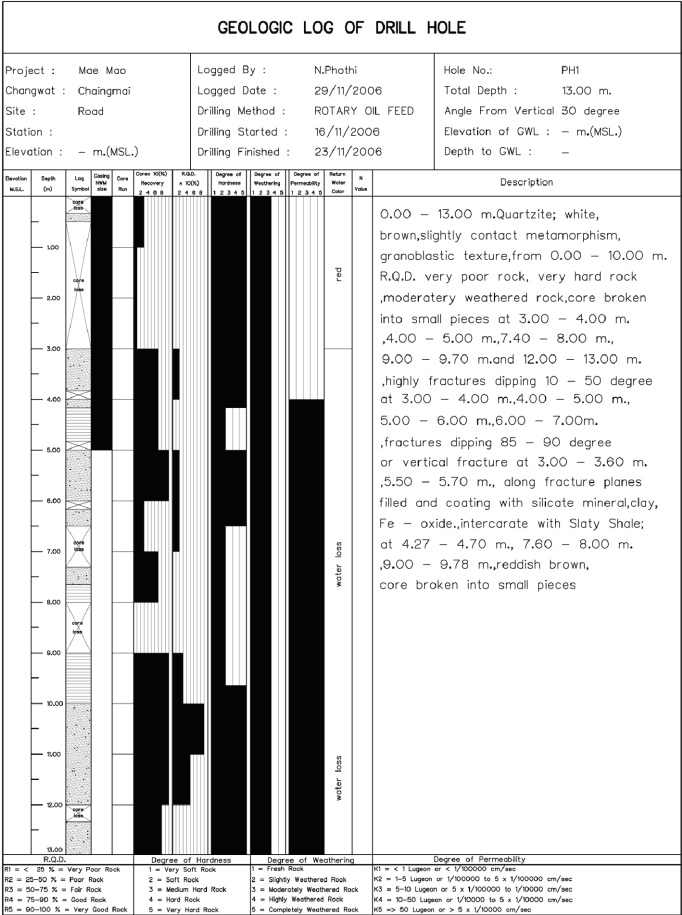


Figure 13: Result of seismic refraction survey.

3.3 Three NX-size boreholes by rock coring were done to the depth of between 10- 20 meter. 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 Figures 14 and 15. The results from three boreholes reveal that there is a 3 meter thick weak layer found as shown in Figure 16. This weak layer has a dip angle quite similar to the dip angle of the foliation plane 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 also saw cut joint (ASTM, 2002.). The total of 9 samples were performed and the results are shown in Table 1 and Figure 17.



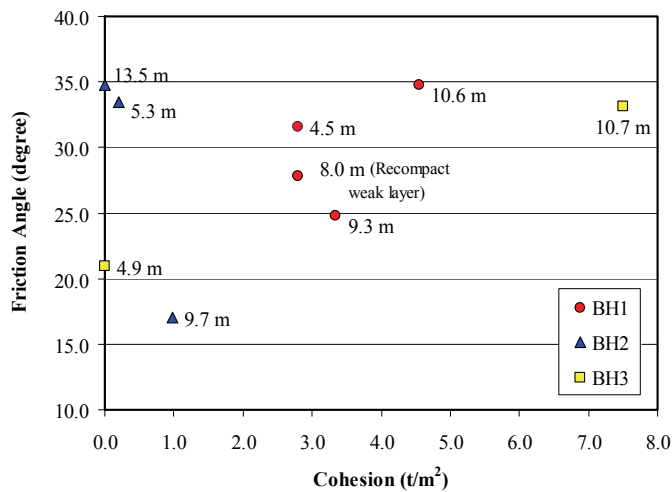


Figure 17: Result of direct shear test.

3.4 Rock mass permeability was determined using packer test (Hoek and Bray, 1981) in order to obtain information for underground 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.

Table 2: Result of field permeability test.

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

4 DESIGN CONCEPT

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

4.1 Rock nail is recommended instead of rock anchored since highly weathered and fractured rock was found in which the tension in the anchored bar might not be able to archived during the installation and the stressing effect might be limited in the area vicinity to the anchor rod.

4.2 The concept of increasing resisting force by counter weight berm in the toe of cut slope and use the upper portion of the hill is omitted in order to prevent the progressive failure since the rock mass in the lower portion was found to be in higher weathering degree. This is according to the result of seismic refraction and visual geological survey.

4.3 The underground water drainage of the slope is extensively applied especially to reduce the pore water pressure near the weak plane layer.

5 STABILITY ANALYSIS AND DESIGN

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

5.1 Back analysis was done with configuration as shown in Figure 18. The 5 meter thick layer of Phyllitic sandstone is modeled together with the 0.5 meter 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 found that the thickness of rock mass layer has to be at least 10 meter thick in order to make the factor of safety of slope less than unity (Figure 19). This back analysis scheme is based on the assumption that the ground water 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 10 meter with almost 30 degree dip angle.

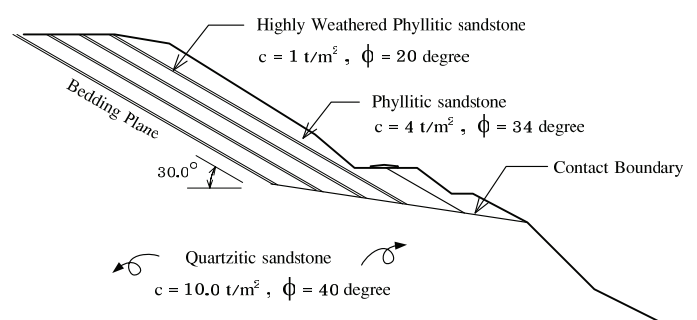


Figure 18: Configuration and properties of slope for back analysis case (Section A-A).

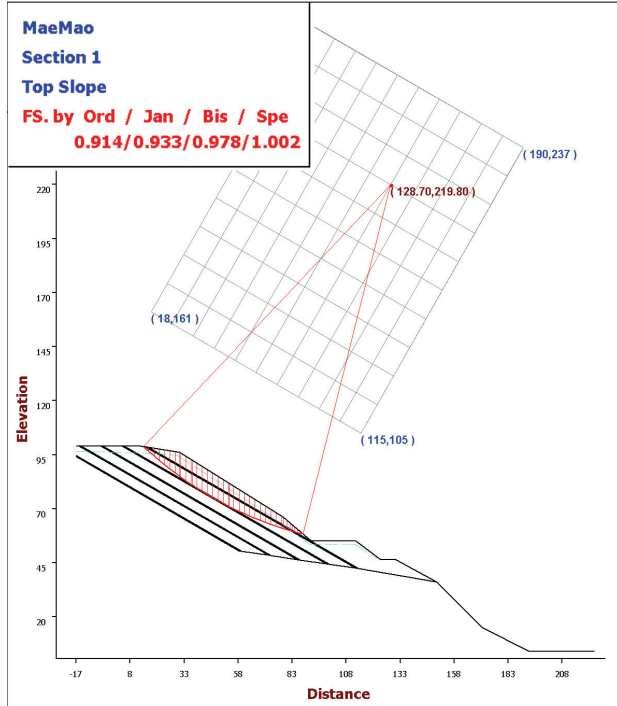


Figure 19: Result of slope stability analysis.

5.2 Analysis for slope remedial design was done based on the design concept explained earlier. The Upper portion slope is designed to be flattening to 30 to 38 degree with 5 meter wide benching steps, 10-12 meter long rock nails and horizontal drains are used as shown in Figure 20. The design cases are shown in Table 3. The ground water level is assumed to be at the ground surface and the horizontal seismic coefficient is used to be equal to 0.05g. The target factor of safety of 1.5 is used. The results of all design cases are shown in Table 4. As for

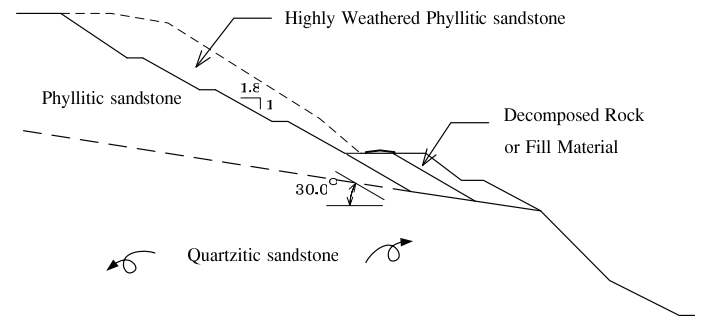


Figure 20: Configuration of remedial work (Section A-A).

Table 3: Remedial cases.

No.	Slope (Ver:Hor)	Rock Nail (W x H, m)	Horizontal Drain (W x H, m)
1	1 : 1.3	2.0 x 2.0	5.0 x 4.0
2	1 : 1.5	2.5 x 2.5	5.0 x 4.0
3	1 : 1.8	-	5.0 x 4.0

Table 4: Stability analysis results.

No.	Method				Seismic Coefficient	Factor of Safety	
	Cut Slope		Rock Nail			Benching	Benching + RockNail
	Slope	Cutting Volume (m³)	Dimension (m x m)	Row			
1	1:1.3	147,575	2.0 x 2.0	24	0.05	1.20	1.50
2	1:1.5	197,020	2.5 x 2.5	14	0.05	1.44	1.50
3	1:1.8	245,150	-	-	0.05	1.73	-

Table 5: Comparison of construction cost.

No.	Construction Cost (Million Baht)						Total Cost
	Cut Slope	Rock Nail	Drainage System	Erosion Control	Miscellaneous Works	Overhead and Profit	
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/m³ Rock Nail = 1,000 baht/m
Horizontal Drain = 1,071 baht/m Erosion Control = 80 baht/m²

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 5). Therefore this schematic was selected as the typical section of the remedial work. It also can be seen from Table 4 and 5 that using rock nail is not quite cost effective solution since the gaining of factor of safety is small comparing to the additional cost that will have to be invested. Figures 21 and 22 shows the typical section, drainage and surface protection drawings.

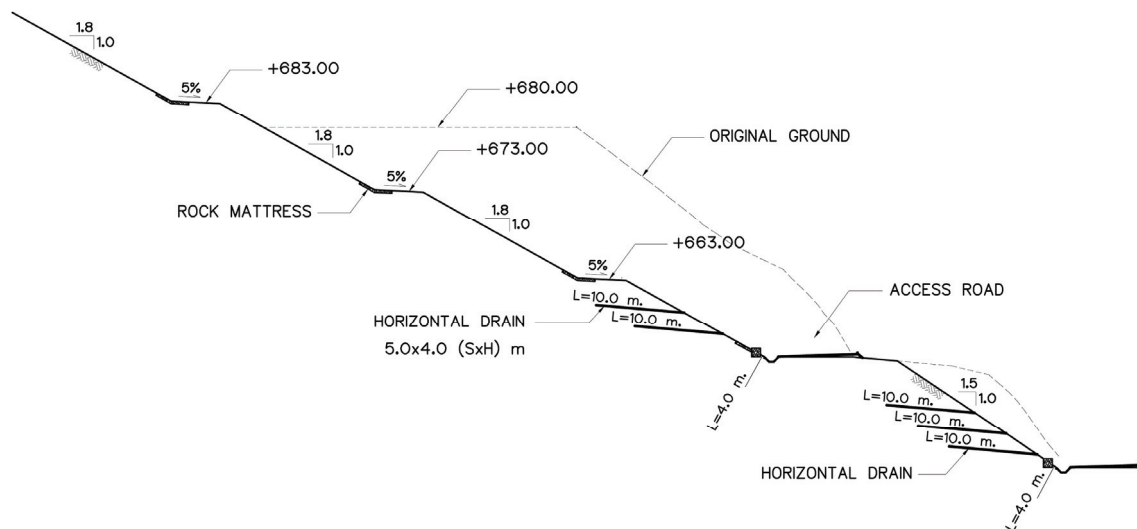


Figure 21: Typical section of repair work (Section A-A).

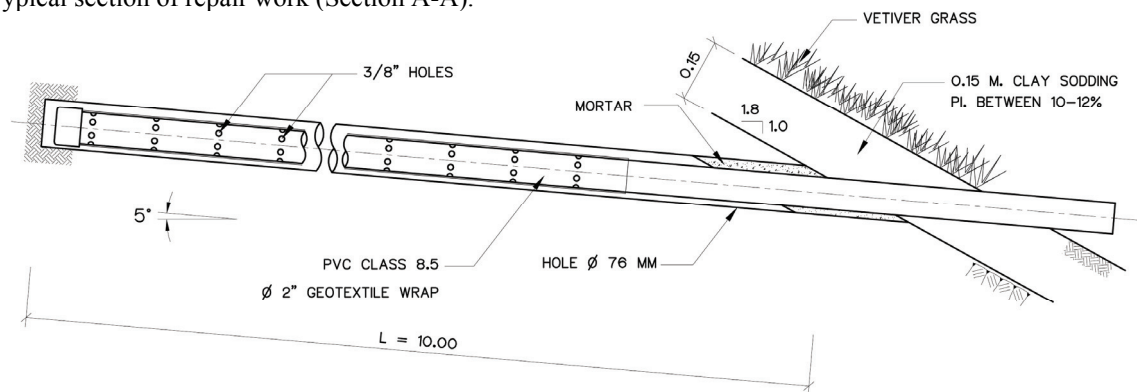


Figure 22: Typical of drainage and erosion control.

6 CONCLUSION

Weak layer is found within the Phyllitic sandstone, this weak layer is suspected to be the cause of the failure of the access road to Mae Mao hydropower project. Back analysis of slope stability matched the actual failure condition. It is important to emphasize the necessity of the comprehensive geological and geotechnical survey before making the decision of the repair work. In this particular case, it's found that the using of rock nails contribute not so cost effective in repair work. Simple cut and benching method is recommended.

ACKNOWLEDGEMENTS

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