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The slope stability problem at Mae Moh Lignite Mine, Lampang Province, Northern Thailand

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Abstract: Landslides and slope failures have been recorded at the Mae Moh Lignite mine, Lampang Province, northern Thailand. These failures were found to be of three principle types, namely rock slides, topple rotational slumps and debris flows.

Field and laboratory investigations were carried out to analyse the stability of the present pit slope which comprises mainly of weathered claystones with thin lignite interlayers. The slope stability analyses were performed using the stereographical techniques of Hendron *et al.* (1971), Markland (1972), Hoek and Bray's (1974) stability chart method, and Bishop's (1955) simplified method of slices for the northwestern slope, and the back-analysis method for the southeastern slope.

Results of analyses indicate that slope instability is controlled by some or all of the following factors; (1) the bedding plane, the preexisting fractures and their concurrent orientation to that of the pit slopes, (2) the geometry of the overall slope and the inclination of the individual benches, (3) the water seepage in the rainy seasons, (4) the degree of weathering and other engineering properties of the rocks.

INTRODUCTION

Lignite is a very important electric energy source in Thailand and is used to generate about 10 percent of the electricity required each year. The only electricity generating enterprise, the Electricity Generating Authority of Thailand (EGAT) owns several lignite deposits for such purpose. The most important and largest deposit is at the Mae Moh lignite mine which is located in Ban Mae Moh, Changwat Lampang, in northern Thailand.

The lignite and associated claystones which belong to the Mae Moh Basin here dip gently to moderately to the north-west from the mining area to the central part of the basin. As excavation progressed down-dip, slope failures occurred in the claystone units overlying the lignite seams. These were wedge failures on the northwestern pit slope and down-dip block slides on the southeastern one. The wedge failures were controlled mostly by preexisting fractures in the clastic rocks while the block slides occurred along the bedding planes when the supporting toe of the slope was removed.

GEOLOGY OF MAE MOH BASIN

Gardner (1967) and Piyasin (1972) who studied the regional geology concluded that the lignite seams in this area belong to the Mae Moh Group, a typical non-marine

Tertiary sedimentary unit which was graben-deposited. The rock group underlies the flat rolling terrain of Nam Mae Moh valley. The rugged, northeast-trending subparallel hill chains which bound the valley floor are composed of the older Triassicc Lampang Group. The sedimentary strata of the Mae Moh Group were also deformed to become an elliptical, NE-trending, open structural basin (Figure 1) whose southwestern end was covered by a younger basalt flow. The mining activity is on the southeastern flank of this syncline whose attitude is rather uniform, striking north-northeast and dipping approximately 25 degrees to the northwest.

A brief review of the stratigraphic sequence (Figure 2) reveals that the Mae Moh Group is subdivided into several rock units. These are, from the bottom to the top, the Lower Claystone, Lignite Q Seam, Interburden Claystone, Lignite K Seam, Overburden Claystone and the Red Bed. The last one is merely a weathered claystone unit. The lignite production is mainly from the lignite K Seam and the known slope failures occurred in the Overburden Claystone and partially in the Red Beds whose occurrence is limited.

Above the Tertiary rocks lies the Quaternary Mae Taeng Formation (Piyasin, 1972) composed of the unconsolidated, loose and pervious gravelly sands. Its role in the slope stability problem is limited.

HISTORY OF LANDSLIDES

The mine pit started at a locality on the southeastern flank of the syncline and progressed down-dip to the northwest toward the central part of the basin, above which 3 operating power plants are located (Figure 3). At present, the rim of the pit is only 400 metres from these plants. As the mine progressed, pit slope failures occurred expecially on the northwestern and southeastern slopes. The failure types were rock slides—circular and wedge failures and planar slides, topple rotational slumps, and debris flows.

A large slope failure which caused concern took place before and during this investigation on a portion of the northwestern pit slope in front of the power plants. In 1979 the slope was cut and shaped into a 3-bench slope, the benches were 8–10 m wide, 10–20 m high with a 35–55° inclination, thus making the overall slope height of 40 m with 200–220 m length and a 30° inclination.

In March 1980 the first wedge failure (Figure 4) was observed on the lower bench by Pramote Pornrattanapitak, a geologist of EGAT (1981, personal communication). The wedge was 18 m high and 20 m long and was controlled by a fault striking NNW and dipping 35° NE which intersected an irregular vertical joint trending ESE.

The wedge grew larger with the help of alternating wet and dry seasons. In March 1981, during this investigation, further sliding of the wedge was noted along the intersection line of the above mentioned discontinuities (Figure 5).

In the following month, April 1981, the pit floor in front of the failure area was excavated for the lowest fourth bench. The bench was 8–10 m high, 3–5 m wide, with

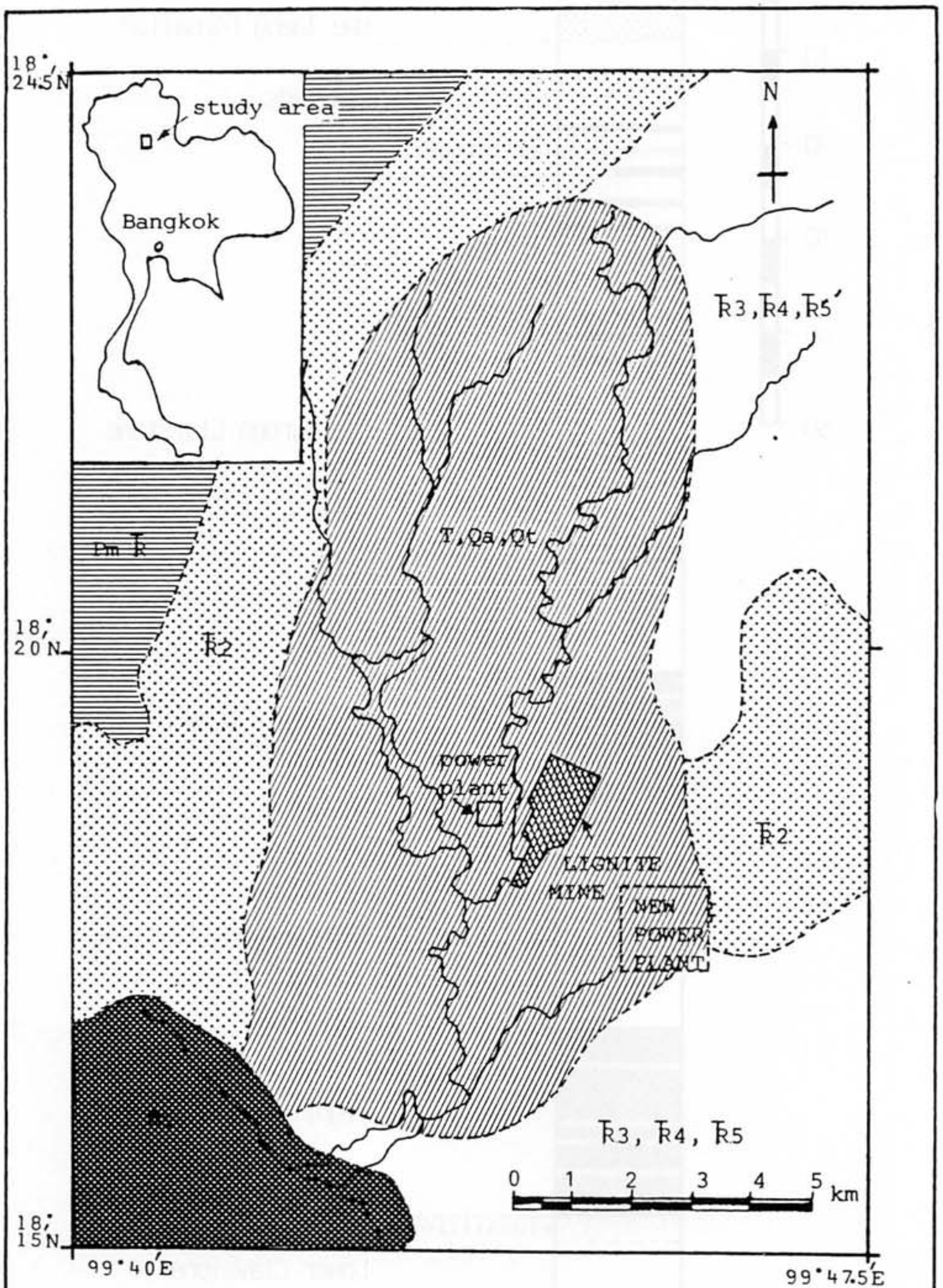

Fig. 1. Regional geology of Mae Moh Basin (simplified from Longworth-CMPS, 1981). The rock formations stated are: Bs-Basalt (Pliestocene); T, Qa, Qt- shale, lignite, terrace gravel and alluvium deposits (Tertiary to Recent); R̄ 3, 4, 5- Shale, sandstone, limestone, mudstone (Hong Hoi, Doi Chang and Pha Daeng Formations; Triassic); R̄ 2- Limestone (Pha Kan Formation; Triassic); Pm R̄- Tuff, agglomerate (Permo-Triassic).

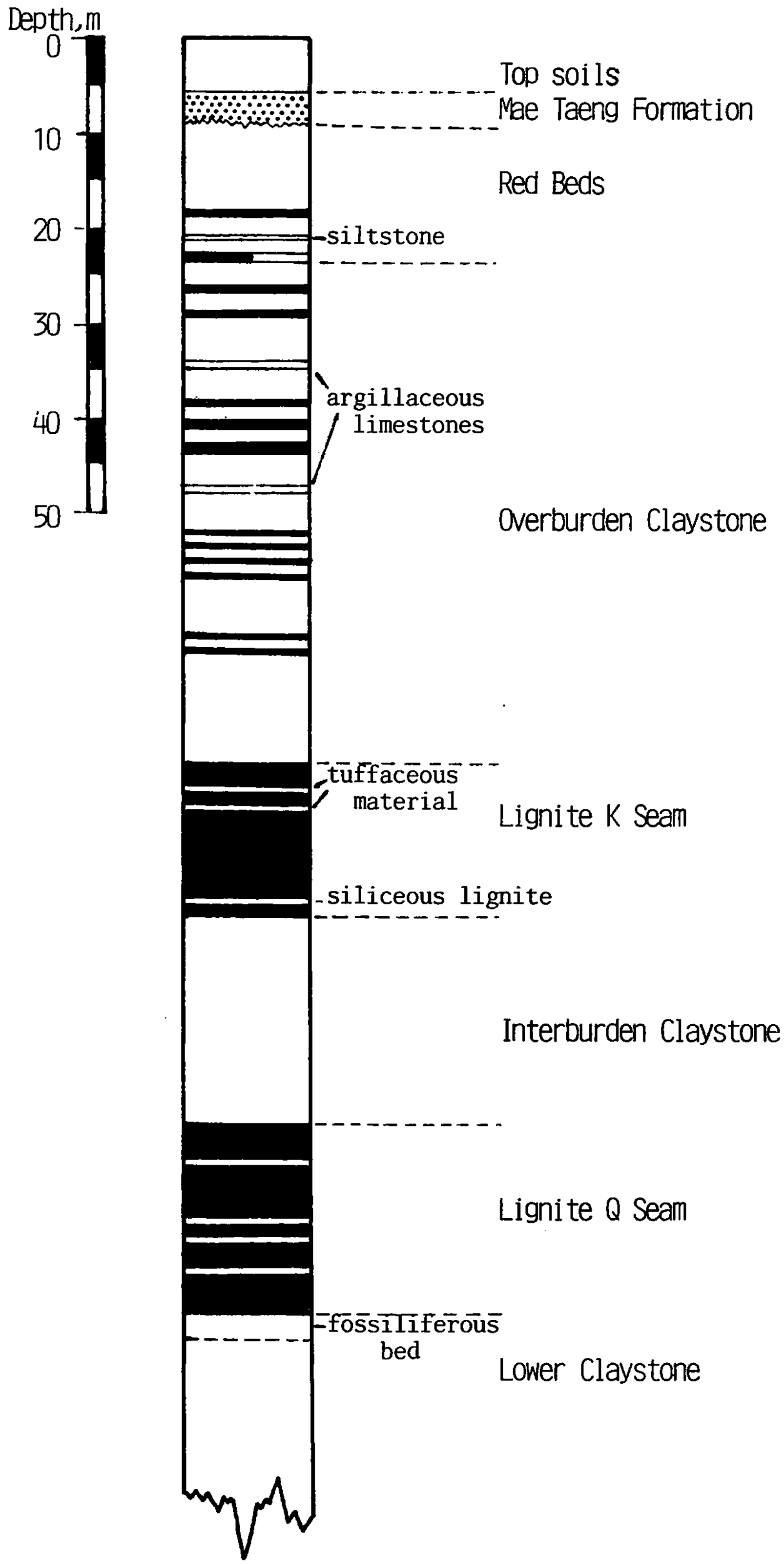

Fig. 2. Stratigraphic column at Mae Moh lignite mine (measured on line N9, mine grid). Black layers are lignite layers, White layers are claystones unless stated otherwise.

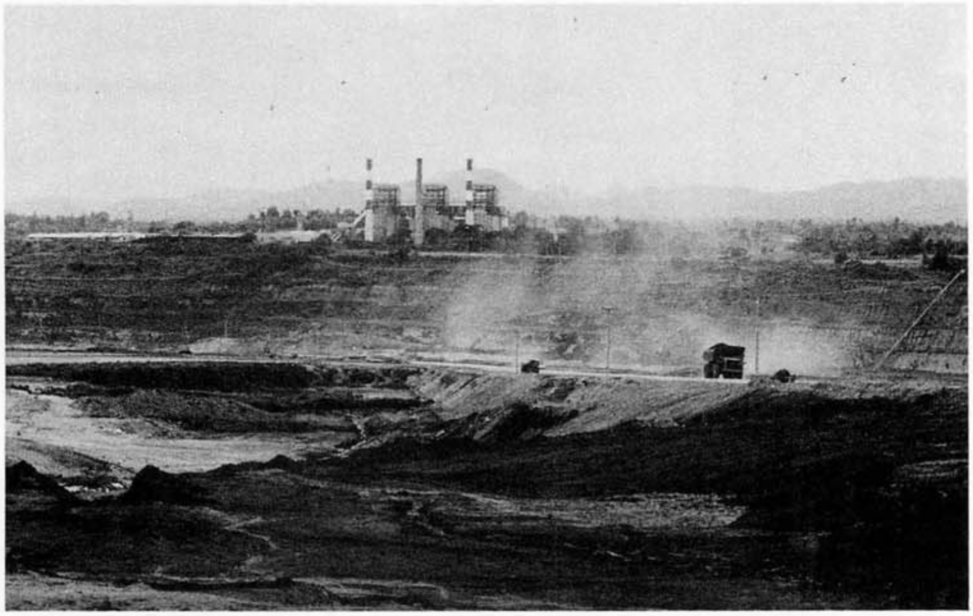

Fig. 3. Mae Moh lignite mine, northwestern pit slope and 3 operating power generators.

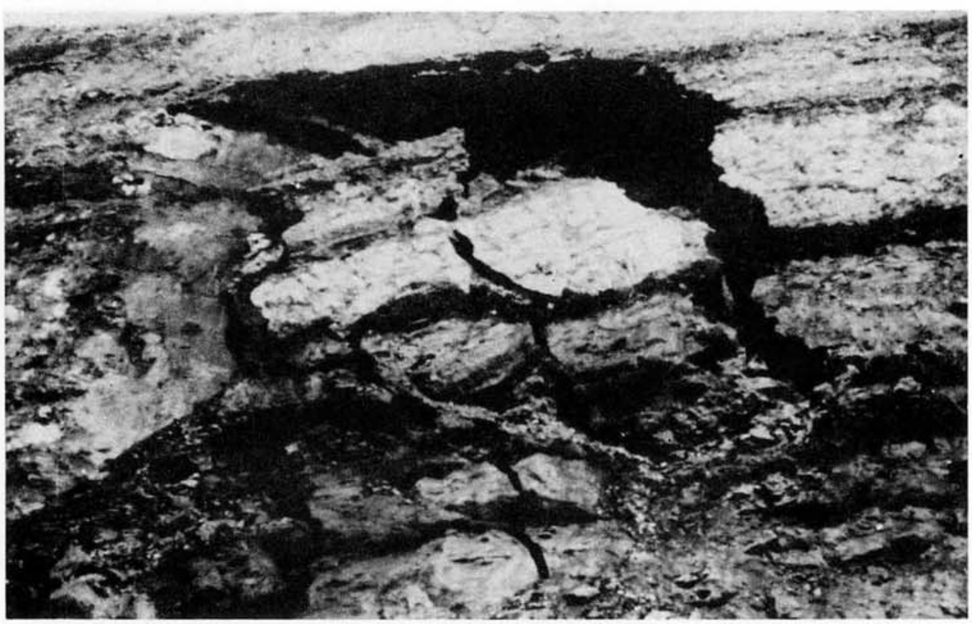

Fig. 4. A wedge failure on the northwestern pit slope in March 1980 (Courtesy of Mr. P. Pornrattanapitak of EGAT).

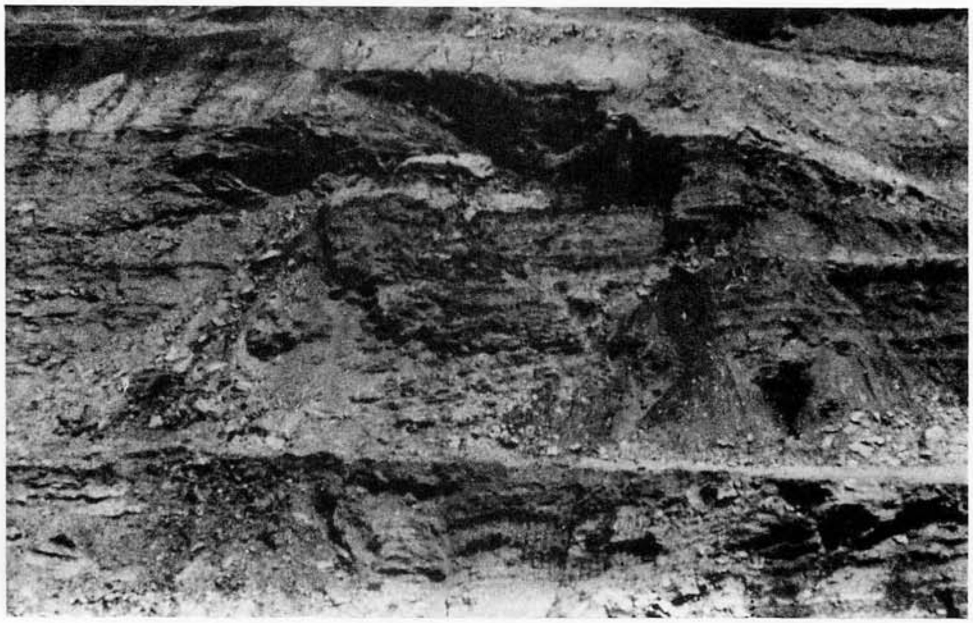

Fig. 5. The continuation of the 1980 wedge failure (Figure 4) as observed in March 1981.

Fig. 6. A major land slide which followed the wedge failure (Figures 4 and 5) had occurred in August 1981.

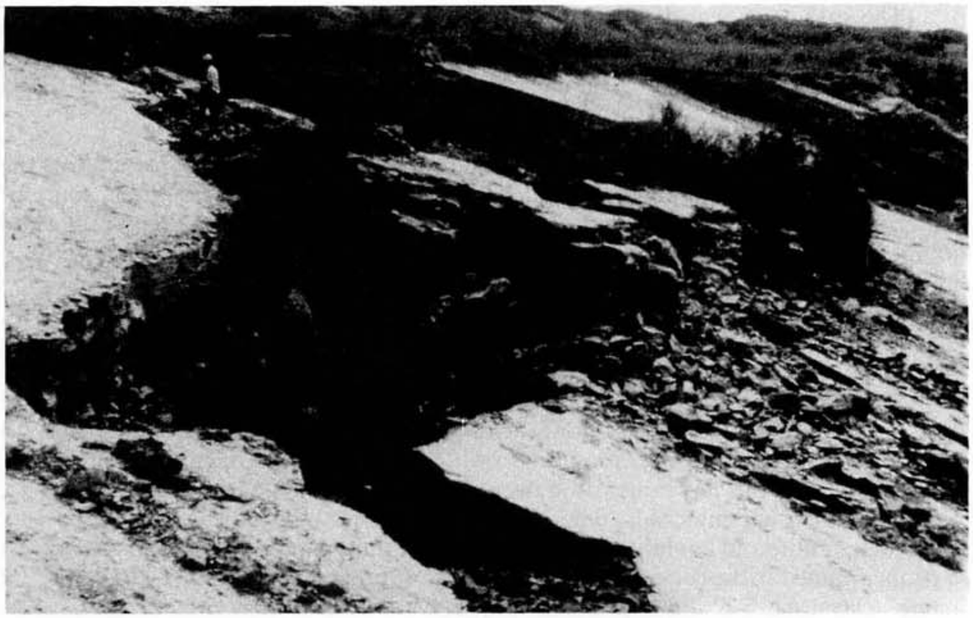

Fig. 7. The planar slides on the southeastern slope. The sliding blocks move along the bedding planes which dip gently into the mine pit.

an 80–85° inclination. This removal of the supporting toe of the slope, caused a further failure.

In July 1981 two tension cracks occurred on the surface behind the slope. The cracks were 30 and 55 m long respectively with their trend parallel to the slope length.

The following months were well into the rainy season. At the peak of the season, August 1981, a major landslide took place. The sliding mass was found to be of a circular failure type. The mass was about 250 m long along the slope face (Figure 6).

On the opposite side of the pit where the bedding planes dip gently into the mine, i.e., transecting the northwestern-facing slope, planar slides are very common (Figure 7). The sliding blocks, sometimes as huge as 400 m^3 move downdip along the bedding planes of the claystone with the help of the joints which are perpendicular to the bedding planes and strike either perpendicular to or parallel to the slope trend.

RESULTS OF AREAL INVESTIGATION

To consider the stability of the pit slopes, several slope portions, both stable and unstable, were chosen for comparison of the results of the analyses. The results of the study it was hoped would give the condition of stability and of the possibility of short and long-term failures.

The study area was divided into 5 representative subareas. Subarea 1 was on the southwestern pit slope, Subareas 2 to 4 on the northwestern slope, and Subarea 5 on the opposite southeastern slope. Subareas 3 and 5 were the known unstable slope portions.

The discontinuity study revealed that, besides the bedding plane, (of the claystones which allows for sliding along itself), there are other fractures, some of which may have played a role in the wedge failures. These fractures are both faults and joints. The faults are mostly of the en echelon normal type, trending to the northeast, north-northeast, and northwest with the moderate to vertical dip. The steeply-dipping joints are mostly close-spaced extensional ones, trending east, northeast, and north.

RESULTS OF MATERIAL INVESTIGATION

Petrographically, the claystone are very fine-grained, mostly clayey size to subordinate silt size. The rocks contain a rather high proportion of calcium carbonate, up to 50%, with the clay minerals make up 30–40%, while the rest consist of quartz, feldspar and other materials. The X-ray diffraction study indicated that the clay minerals are illite and kaolinite without montmorillonite. The mineral contents suggest a proper name for the rock as "marlstone" (according to Lewan, 1978). However, the name "claystones" is still used in this paper.

The claystone specimens from Subareas 1 to 5 were collected for the determination of the density, water content and the direct shear strength tests. The intact shear strength parameters of the fresh claystones are significantly greater in value than those of the slightly to moderately weathered rocks as expected especially for the naturally-moist condition (Table 1). The parameters include the peak and residual values. In the saturated condition, however, the results do not follow this finding, and many specimens crack during the process of saturation that they could not be used to perform the test. But as a whole, the incomplete results illustrate a tendency that the shear strength parameters of the naturally-moist samples have a higher value than those of the saturated rocks.

The direct shear strength test was also performed on the samples with the smooth bedding planes and with the naturally rough fracture surface. The test results in Table 2 indicate that the strength parameters on the smooth bedding planes are generally lower than those on the rough fracture surfaces, and that those of the fresh to slightly weathered rocks are higher than those of more weathered rocks.

Tests on the Red Bed rocks were not performed in this study as they had been studied before by LONGWORTH-CMPS Engineers (1981). Their results generally conform with the findings of the Overburnden Claystone rocks of this study.

ANALYSIS OF SLOPE STABILITY

The geometry of the slope and the sets of discontinuities in each subarea were also used in the slope stability analyses, using both soil and rock mechanics approaches. In this study, the possibility of circular failure, wedge failure and planar sliding were

TABLE 1

INTACT SHEAR STRENGTH TEST RESULTS OF FRESH (FR) AND SLIGHTLY- TO MODERATELY WEATHERED (SW-MW) CLAYSTONES.

Sample no.	Sample description	Shear strength parameters									
		Natural condition					Saturated condition				
		C	ϕ_p	C_r	ϕ_r	w	C_p	ϕ_p	C_r	ϕ_r	w
CL-1	Claystone, Fr	28.9	29.2	1.8	18.0	20.0	23.0	21.9	1.2	21.4	23.4
CL-2	Claystone, Fr	5.8	52.7	2.2	15.1	15.7	17.0	12.5	2.0	17.0	22.9
CL-3	Claystone, Fr	31.6	15.5	3.3	16.2	19.3	–	–	–	–	23.2
CL-4	Claystone, Fr	25.0	30.9	0.8	29.2	19.2	14.1	25.8	1.5	17.4	21.1
CL-5	Claystone, Fr	25.7	21.8	1.4	20.8	19.5	–	–	–	–	23.2
CL-6	Claystone, Sw-Mw	6.8	32.7	0.0	24.4	20.3	9.0	13.2	0.5	12.0	23.5
CL-7	Claystone, Sw-Mw	22.7	18.5	3.0	17.2	19.8	–	–	–	–	24.0
CL-8	Claystone, Sw-Mw	8.2	25.6	4.5	25.0	20.2	3.2	24.0	0.0	20.5	23.8
CL-9	Claystone, Sw-Mw	8.0	19.6	1.1	10.0	19.5	7.8	31.7	2.3	27.1	22.9

Notations:
C_p = Peak cohesion (Kg/cm^2)
C_r = Residual cohesion (Kg/cm^2)
ϕ_p = Peak internal friction angle (degrees)
ϕ_r = Residual internal friction angle (degrees)
W = Water content (%)

TABLE 2

DEFECT SHEAR STRENGTH TEST RESULTS OF CLAYSTONES. DEGREE OF WEATHERING IS THE SAME AS IN TABLE 1

Sample no.	Sample description	w (%)	Defect types	Shear strength C_r(kg/cm^2)	parameters ϕ_r(degree)
CL-D1	Claystone, Fr-Sw	16.0	Bedding plane, smooth	0.50	17.0
CL-D2	Claystone, Fr-Sw	18.5		0.72	15.8
CL-D3	Claystone, Fr-Sw	18.6		0.75	16.0
CL-D4	Claystone, Sw-Mw	17.5		0.30	15.7
CL-D5	Claystone, Sw-Mw	19.0		0.57	15.9
			Average	0.57	16.1
CL-D6	Claystone, Fr-Sw	14.3	Jointing plane, rough	1.05	24.0
CL-D7	Claystone, Fr-Sw	16.7		1.45	20.2
CL-D8	Claystone, Fr-Sw	18.3		0.90	24.0
CL-D9	Claystone, Sw-Mw	15.6		0.91	21.8
CL-D10	Claystone, Sw-Mw	17.5		0.84	20.6
			Average	1.03	22.1

TABLE 3

RESULTS OF SLOPE STABILITY ANALYSES USING THE STEREOGRAPHICAL METHODS.

Subarea	Slope angle		RESULTS OF SLOPE STABILITY ANALYSES: Markland's technique		Hendron *et al.*'s technique		Adjustment of slope angle and comments
	Overall	Individual bench	Stability condition and sliding direction	Plane of weakness caused unstable	F.S.$_{max.}$ at $\phi = 22°$	F.S.$_{min.}$ at $\phi = 16°$	
1	16°	26°	Stable	None			Not necessary
2	35°	50°	Unstable N 88°E/50°	J(62/68 SE) and J(308/75 NE)	0.93	0.63	Decrease 2–4° for individual bench slope.
3	33°	50°	Unstable S 72°E/37°	J(108/885) and F(345/42 E)	0.67	0.47	Decrease 5–10° is possible for individual bench slope.
			S 71°E/45°	J(108/885 and F(340/52 E)	0.79	0.56	
4	38°	55°	Unstable S 24°E/38°	F(340/85 E) and F(139/70 SW)	1.89	1.33	Stability condition due to F_1 and F_2 is considered to be stable.
5	24°	40°	Unstable N 70°W/20–25	B(202/20 W)	1.11	0.79	Decrease the slope angle is not possible in the working area.

Notations: J, F, B (202/20 w) = Joint, fault, and bedding plane with values of strike and dip in parenthesis.

considered while other types of the down-slope movements, for example, the topple slumps and the debris flows received no attention as they are unpredictable. The circular failure is controlled by the low cohesive strength in the high-degree weathered rocks and the jointed less-weathered claystone. The wedge failure is controlled by the intersection line of the major fractures which emerges on the slope face, and the planar sliding by the discontinuities plus the bedding plane which dips at a smaller angle in the direction of the slope inclination.

In some analysis methods the water condition in the subareas was considered. Water regularly seeps into the permeable rocks causing a higher water pressure and lubricate the sliding blocks. This occurs regularly in the rainy season. In this analysis, water saturated conditions ranging from the fully-drained or dry to fully-saturated conditions were assumed.

TABLE 4

RESULTS OF SLOPE STABILITY ANALYSIS USING THE CIRCULAR FAILURE CHARTS METHOD FOR MINIMUM/MAXIMUM C, ϕ VALUES

Subarea	Cross-section	Slope	Average factor of safety for		
			Dry slope	Partially sat	Fully sat.
1	a	1	4.70/8.15	3.91/6.69	3.72/6.32
		2	1.77/2.72	-/-	1.21/1.84
		3	1.23/2.65	-/-	1.19/1.82
2	a	1	1.68/2.79	1.55/2.63	1.43/2.43
		2	1.28/2.08	1.08/1.80	1.00/1.66
		3	1.22/1.87	0.93/1.40	0.60/1.34
	b	1	1.83/3.10	1.72/2.96	1/60/2.65
		2	1.25/2.04	1.08/1.83	0.99/1.66
		3	1.03/1.28	0.76/1.20	0.46/1.11
3	a	1	2.31/3.83	2.08/3.51	1.92/3.30
		2	1.60/2.64	1.32/2.21	1.23/2.06
		3	1.02/1.71	0.79/1.27	0.72/1.15
	b	1	1.30/2.19	1.21/2.05	1.09/1.84
		2	1.30/2.13	1.10/1.84	1.01/1.68
		3	1.04/1.72	0.81/1.38	0.74/1.26
4	a	1	1.78/3.05	1.68/2.92	1.57/2.69
		2	2.11/3.73	1.73/2.92	1.62/2.73
		3	1.18/1.87	0.89/1.42	0.83/1.35
	b	1	2.23/3.77	2.09/3.60	1.87/3.37
		2	1.32/2.14	1.15/1.91	1.04/1.72
		3	0.91/1.44	0.71/1.15	0.63/1.03

Notations:- Slope 1 = Single bench slope.,
2 = Two bench slope.,
3 = Overall slope.

Methods of analysis include stereographical methods using an equal-area and equal-angle stereographical projection as developed by Markland (1972) and Hendron *et al.*, (1971) respectively, Hoek and Bray's (1974) stability charts, the Simplified Bishop method of slices, and the back analysis for the planar sliding. The stereographic methods and the stability charts give a quick view of the possibility of failure while the other methods give more detailed results.

The results of the slope stability analysis using the stereographical methods are summarized in Table 3. They indicate that Subarea 1 is stable while Subareas 2 to 4 are prone to wedge failure and Subarea 5 to planar sliding.

The circular failure chart method results (Table 4) indicate that the factor of safety decreases from a single-bench to the overall slope conditions, and from dry to fully-saturated conditions. In some cases the factor of safety is below unity indicating

TABLE 5

RESULTS OF SLOPE STABILITY ANALYSIS USING SIMPLIFIED BISHOP METHOD OF SLICES

Subarea	Cross-section	Groundwater condition	Assumed r_u	Factor of safety (F.S.)	
				$F.S._{max}$	$F.S._{min}$
	a	Partially	0.10	1.86	1.21
		sat.	0.24	1.70	1.09
		"	0.34	1.54	0.95
		Fully sat.	0.49	1.26	0.79
2	b	Partially	0.10	1.65	1.06
		sat.	0.24	1.50	0.96
		"	0.34	1.30	0.82
		Fully sat.	0.49	1.09	0.67
	a	Partially	0.10	1.67	1.07
		sat.	0.24	1.53	0.98
		"	0.34	1.33	0.83
		Fully sat.	0.49	1.13	0.70
3	b	Partially	0.10	1.86	1.30
		sat.	0.24	1.71	1.05
		"	0.34	1.50	0.91
		Fully sat.	0.49	1.28	0.75
	a	Partially	0.10	1.90	1.20
		sat.	0.24	1.75	1.10
		"	0.34	1.55	0.89
		Fully sat.	0.49	1.34	0.81
4	b	Partially	0.10	1.49	0.95
		sat.	0.24	1.36	0.85
		"	0.34	1.17	0.73
		Fully sat.	0.49	0.98	0.60

inequilibrium conditions, hence the slopes are unstable in these cases. In general, the overall slope has a higher chance of failure especially when partially to fully-saturated than the single- or two-bench condition.

The results of Simplified Bishop method of slices in Table 5 indicate that the factor of safety decreases with increasing degree of saturation. However, it is not very conclusive whether slope failure would occur in any subarea.

The back analysis for the plane failure was performed particularly for Subarea 5. The results only indicate the conditions of barely-stable to unstable, either for the dry or fully-saturated rocks, or for the individual bench or the overall slope, especially when the slope is cut with the inclination at a higher angle than the dip of the claystone beds.

CONCLUSION

This study shows that the slope stability problems at Mae Moh lignite mine are controlled by the orientation of the claystone beds, the orientation of the faults and major joints and their intersection, the orientation, critical height and critical angle of the overall cut slopes and individual benches, the degree of weathering and the blocky-nature of some claystones, and the degree of water saturation in this area.

ACKNOWLEDGEMENTS

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