Four Landslide Investigations in Malaysia

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Abstract: Four case histories of landslides in three cut slopes and one filled slope in Malaysia with underlying formations of weathered soils derived from igneous rocks and meta-sedimentary formations have been investigated and will be presented in this paper. Investigating processes consisting of site reconnaissance, topography survey, subsurface investigation, laboratory testing, back-analyses, instrumentation scheme for slip surface detection and groundwater regime establishment will be briefly discussed. Rainfall records spanning the period before landslide incident till the end of investigation were gathered to reveal the relationship between the landslide and the rainfall. Comparisons have also been carried out between the laboratory strengths interpreted from consolidated isotropically undrained triaxial tests and direct shear box tests and the back-analysed strengths from both conventional limit equilibrium stability analyses and finite element analyses. The c-ϕ strength reduction method is used in the finite element analyses for back-analysing the mobilized strength of slope stability. In some case histories, finite element analyses have successfully demonstrated the mechanism of progressive failure in the case study with high cut slopes, in which stress distribution within the slope body is highly non-uniform, particularly at the developed slip surface. Comprehensive instrumentation was implemented at certain sites to provide valuable information for investigation and to offer insight of the post-failure behaviour of the distressed slopes. The back-calculated residual strengths of the failed slope are somehow deviated from the common residual strength correlation with the liquid limit and the clay size fraction.

1 INTRODUCTION

1.1 *General*

It has always been conveniently and mistakenly assumed that cut slope possesses higher shear strength and therefore could sustain steeper cutting gradient. In Malaysia, cut slope gradient as steep as 1V:1H has been commonly adopted. Though many steep cut slopes sustain without failure which is mostly like due to the existence of suction, cut slope failure has topped the list of slope failures. The inherent variation of soil properties, geological structures, weathering profile and the possible existence of a localised layer of weak material (eg. clay seam) could contribute to the failure of cut slope. Therefore, it is imperative that observation of the variation of weathering profile, geological features and other characteristics of soil is carried out during the cutting process so that the design could be reviewed from time to time.

2.0 *Cut Slope in Johore*

The slope failure occurred after a heavy downpour involved a two-berm cut slope (gradient 1V:1.5H), which was formed for a building construction. A comprehensive geotechnical investigation had been carried out to investigate the causes of the failure and to propose remedial measures.

2.1 *Topography and Geological Conditions*

The site is located on a relatively high ground with original reduced level ranging from RL 54.0m to RL 106.0m over a distance of about 320m. The mineral resources map of Johor (Geological Survey, 1982) shows that the site is situated at Jurong Formation which is underlain by mainly basic intrusive gabbro and intermediate intrusive rocks such as syenite, tonalite and diorite.

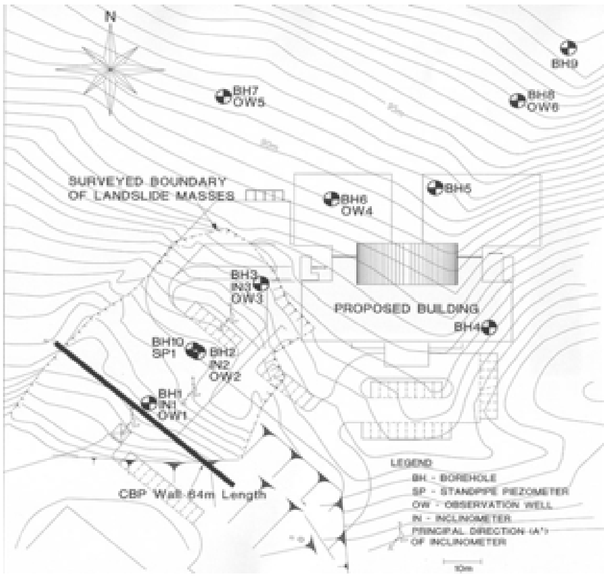

Fig. 1. Borehole and Instrumentation Layout Plan

2.2 *Subsurface Investigation & Instrumentation*

Subsurface investigation and instrumentation consisting of ten boreholes, three inclinometers, six observation wells and one standpipe piezometer were carried out. The layout of the boreholes and instrumentations is shown in Figure 1. A total of three inclinometers, IN1, IN2 and IN3, three observation wells, OW1, OW2, OW3 and one standpipe piezometer SP1 was installed within the failed mass. The inclinometers were sheared off shortly after the installation. Figure 2 shows the multiple slip surfaces and groundwater level interpreted from the instrumentations.

2.3 *Laboratory Test Results*

A series of laboratory tests has been carried out on the samples obtained from the subsurface investigation works. Only selected test results would be presented below.

Eleven numbers of C.I.U.(Consolidated Undrained Triaxial Test with pore water measurement) have been carried out and the interpreted peak strength and critical strength parameters are c'_p=3.5kPa, ϕ_p'=32^o and c_{cr}'=3.0kPa, ϕ_{cr}'=29^o. Ten numbers of multiple reversal shear box test were also carried out on the reconstituted samples to obtain the residual strength of soil. However, the results of the shear box tests are fairly scattered and range from c'_r=0kPa, ϕ'_r =14^o and c'_r=31.4 kPa, ϕ_r'=21^o.

2.4 *Back-Analysis and Engineering Assessment*

The back-analyses were performed using PC-Stabl (Harald, 1989) a limit equilibrium program and Plaxis (Brinkgrieve, 2002) which is a finite element program. The well-defined slip surface and groundwater table as established in the field investigation were adopted in the back-analysis model. The back analyses of mobilised shear strength at the identified slip surface were performed based on the following two conditions:

a. Original slope profile after cutting of the lower two-berm slopes, but before failure.

b. Slope profile immediately after failure but continue to creep.

To obtain the back calculated mobilized shear strength along the slip surface, the shear strength parameters, c' and ϕ' are adjusted till the Factor of Safety is unity (1.0). The back-analysed shear strength parameters for condition (a) and (b) using PC-Stabl and Plaxis are tabulated as below:

Table 1 Back Analysed Strength Using PCSTABL and PLAXIS

Methodology	Condition (a)		Condition (b)	
	$c'_{(a)}$	$\phi'_{(a)}$	$c'_{(b)}$	$\phi'_{(b)}$
1. Back Analysis (PC-STABL)	0	24^o	0	14.4^o
2. Back Analysis (PLAXIS)	0.5	25.9^o	0.5	15^o

It is expected that the back-analysed shear strength parameters in conditions (a) and (b) correspond to critical state and residual strength respectively. The residual friction angle obtained from the correlation with liquid limit and clay size fraction (Mesri et al. 1986) range from ϕ'=18^o to ϕ'=27^o, which deviates from the back-analysis and shear box test results. Table 2 summarises the shear strength parameters obtained from different methodologies.

Table 2 Back Analysed Strength Using PCSTABL and PLAXIS

Methodology	Residual Strength				Peak Strength			
	Upper Bound		Lower Bound		Upper Bound		Lower Bound	
	c'_r	ϕ'_r	c'_r	ϕ'_r	c'_p	ϕ'_p	c'_p	ϕ'_p
1. Liquid Limit	0	26^o	0	18^o	-	-	-	-
2. Clay Size Fraction	0	27^o	0	21^o	-	-	-	-
3. Reversal Shear Box Tests on Re-constituted Samples	31.4	21^o	0	14^o	39	30^o	5.9	21^o
4. C.I.U. on Undisturbed Samples	Peak Strength Parameters : c'_p = 3.5kPa, ϕ'_p = 32 ^{o}							
	Critical State Strength Parameters : c'_{cr} = 3.0kPa, ϕ'_{cr} = 29 ^{o}							

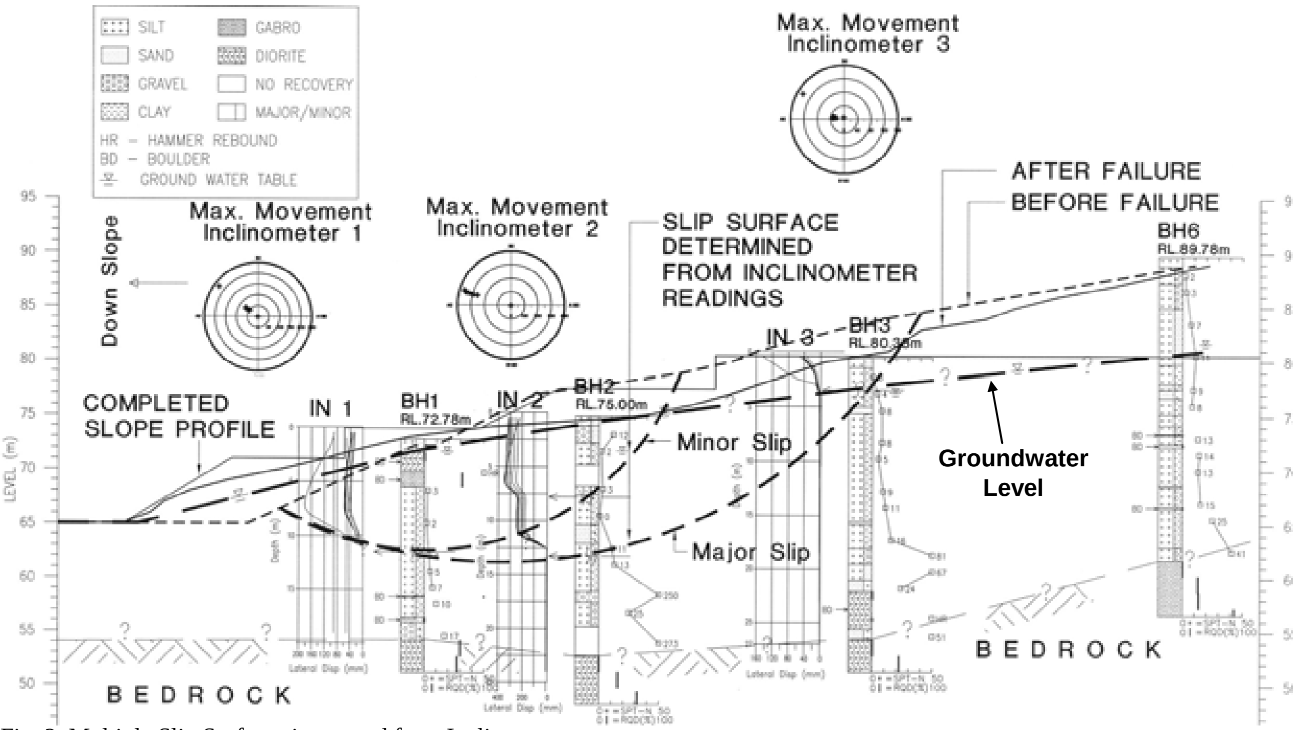

Fig. 2. Multiple Slip Surfaces intepreted from Inclinometers

Based on the laboratory test and back-analysis results, it is credible to deduce that there is an existence of thin layer with low shear strength at the slip surface. The slip surface is difficult to be determine accurately in the subsurface investigation works unless with the help of inclinometer. Such weak layer is believed to have experienced sufficient shearing strain prior to the failure, therefore exhibits the average mobilized shear strength lower than critical state strength.

3.0 *Cut Slope in Kelantan*

The original design of failed slope consists of seven (7) upper berms of 1V:1H cut slope and five (5) lower berms of 4V:1H cut slope with soil nails. A massive slope failure occurred before soil nails were installed at the lowest berm. The slope profiles of original topography and the original design are shown in Figure 3. Figure 4 shows the front view of the failed slope.

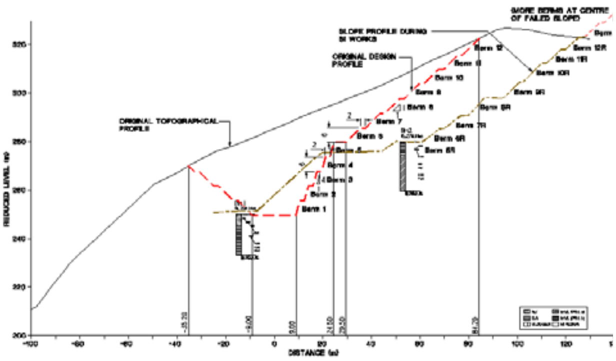

Fig. 3. Original Topography and Design Profiles

3.1 *Topography and Geological Conditions*

The site is located on high ground with reduced level ranging from RL210m to RL330m. The site is underlain by Shale facies, which consists of mudstone and siltstone (Geological Survey Malaysia, 1974).

Fig. 4. Front View of Failed Slope

3.2 *Subsurface Investigation and Geological Mapping*

The subsurface investigation consisted of two boreholes. Three C.I.U tests were performed on samples consisted of Grade III and IV material. The interpreted shear strength parameters are summarised in Table 3.

Table 3 Interpreted Shear Strength Parameters

Weathering Grade	Effective Cohesion, c' (kPa)		Effective Friction Angle, ϕ' (^{o})	
Grade IV	Peak	30	Peak	33
	Residual	0	Residual	33
Grade III	Peak	30	Peak	39
	Residual	0	Residual	33

Geological mapping was also carried out during the subsurface investigation works. It was observed that the site was dry and no water seepage was seen. The joint sets mapped were day-lighting towards the road. Most of the exposed material on the slope suface were Grade III to V. The interpreted subsoil weathering profile is shown in Figure 5. Kinematic analysis was later performed on the collected geological mapping data using rock engineering software (Watts et al.,2003). It can be conceded that plane failure is more probable compared to other modes of failure due to the presence of day-lighting geological structure.

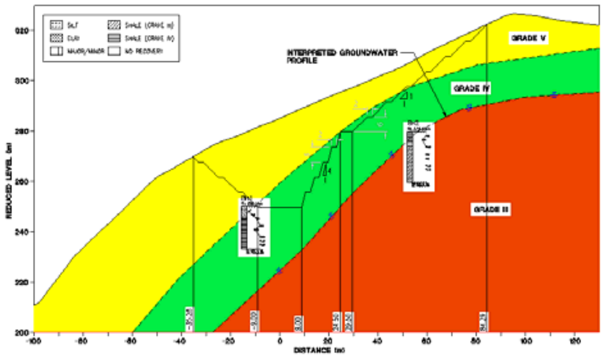

Fig. 5. Interpreted Subsoil Weathering Profile

3.3 *Back-Analysis and Engineering Assessment*

The back-analyses were performed using GGU, a limit equilibrium program (Dr Johann) and Plaxis. As no undisturbed samples was collected for Grade V material, the shear strength parameters of c'_p=5kPa, ϕ_p'=33^o has been adopted. The limit equilibrium analysis indicated that global Factor of Safety is marginally more than 1.0 when the excavation was down to the lowest berm as per design as shown in Figure 6. Besides that, the Factor of Safety for local slope stability at the 1V:1H cut upper cut slopes is also marginally more than 1.0.

The finite element analysis was carried out using typical Mohr-Coulomb strength criteria and elasto-plastic models to simulate various excavation stages and the failure mechanism. The development of plastic points within the soil body during the cutting process indicated the mobilization of peak strength in these soil elements. Eventually a well-defined slip surface was formed when the excavation reached the lowest berm. Figure 7 shows the development of plastic points and slip surface in the finite element model. In short, the analysis demonstrates the phenomenon of progressive failure with the progressive cutting of slope. It was also shown that the installed soil nails failed to provide significant strengthening action to the overall slope due to inadequate nail length in relation to the entire failed slope geometry.

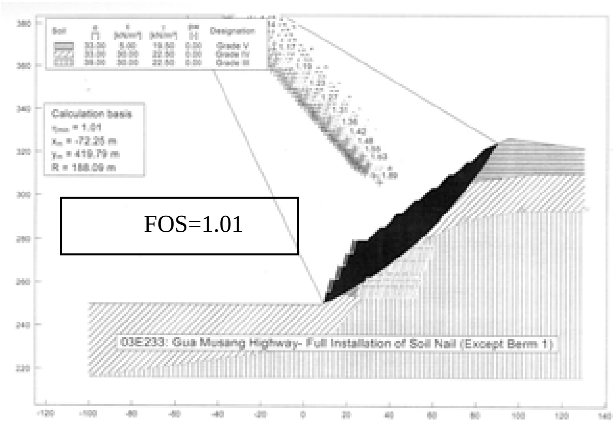

Fig. 6. Analysis Results for Global Slope Stability

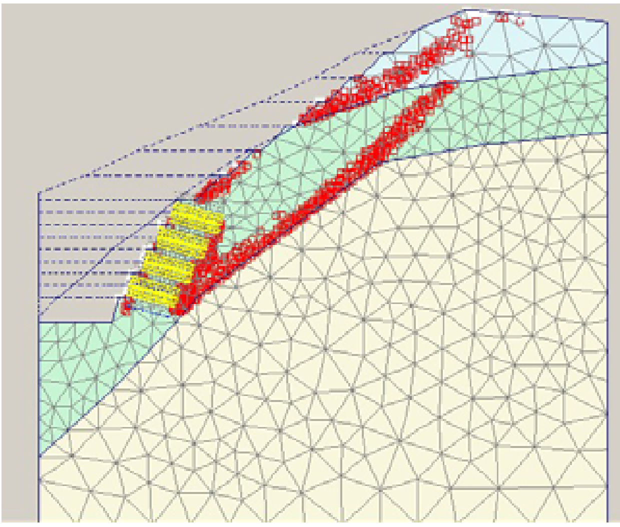

Fig. 7. Development of Plastic Points in Finite Element Model (After cutting of Twelve Upper Berms)

4.0 *Cut Slope in Kedah*

The cut slope involved in the failure was about 27m high (5 to 6 berms with gradient of 1V:1H) and was located beside a proposed road alignment. The failure occurred in two incidents, where a localized stretch of about 50m failed two months after formation of slope. A major failure of the cut slope with 250m length occurred a month after the localized failure following heavy rainfall. Figure 8 shows the front view of the failed slope.

Fig. 8. Front View of Failed Slope

4.1 *Topography and Geological Conditions*

The site is located on a high ground with levels ranging from about RL300m to RL350m. To the east of the slope failure is a mountain ridge with a peak elevation of about RL573m. The site is found to be located within upstream catchment area of a river.

It is interesting to note that complex geological structures and features exist within the region of the site. The site is underlain by granite (Geological Survey Malaysia, 1970). Burton (1970) reported that two sets of fault dominate the structure of the region, which is 323^{o} (northwest-southeast) and 032^{o} (northeast-southwest). Exposed intact granite bedrock was observed at the top of the failed scarp. Prismatic feldspar phenocrysts which are relatively resistant to weathering are found within the granite rock as shown in Figure 9.

Fig. 9. Feldspar Phenocrysts within Granite

4.2 *Subsurface Investigation & Instrumentation*

The subsurface investigation and instrumentation works included three boreholes within the center of failed mass, with two observation wells and one inclinometer installed.

Site observation and investigation revealed that the subsoil is fairly sandy, and is classified as silty sand according to the British Soil Classification System. A slip surface was also detected by the inclinometer installed in the center of the slope. Observation wells were also installed at the toe and top of the failed mass. It was observed that the piezometric level at the slope toe was high, with readings ranging from 0.37m to 2.2m above ground level. The interpreted slip surface and piezometric levels are shown in Figure 10.

C.I.U tests have been carried out and the interpreted peak and critical shear strength parameters are c'_p=2kPa, ϕ_p'=30^{o} and c_{cr}'=1.9kPa, ϕ_{cr}'=28^{o} respectively.

4.3 *Back-Analysis and Engineering Assessment*

The back-analyses were performed using PC-Stabl and Plaxis using interpreted slip surface and piezometric as established in the field investigation. The back analyses of mobilised shear strength at the identified slip surface was found to be are c'_m=0kPa, ϕ_m'=30^{o} which agrees well with the C.I.U results. If the C.I.U results were adopted in a limit equilibrium analysis, the Factor of Safety would be less than unity, even without the effect of piezometric level. Therefore, the temporary stability of the slope after formation could be attributed to the existence of soil suction.

The rainfall record obtained from the Meteorological Department shows that the rainfall amount was in a rising trend after the formation of cut slope. It can therefore be deduced that large rainfall amount had contributed to the increase infiltration and the rise of piezometric level in slope. With the rise of the piezometric level, soil suction and effective shear strength reduce, thus triggering the collapse of slope.

5.0 *Filled Slope in Selangor*

A filled slope covered with gabion mattress was formed over a natural valley as a construction platform to facilitate the laying of an utility pipeline carrying petrochemical products. The filled slope encountered two failures after formation and some remedial works had been carried out. When the third failure event occurred, which was a major failure that occurred after intense rainfall period, the authors were called in to investigate and propose remedial works for the failed slope.

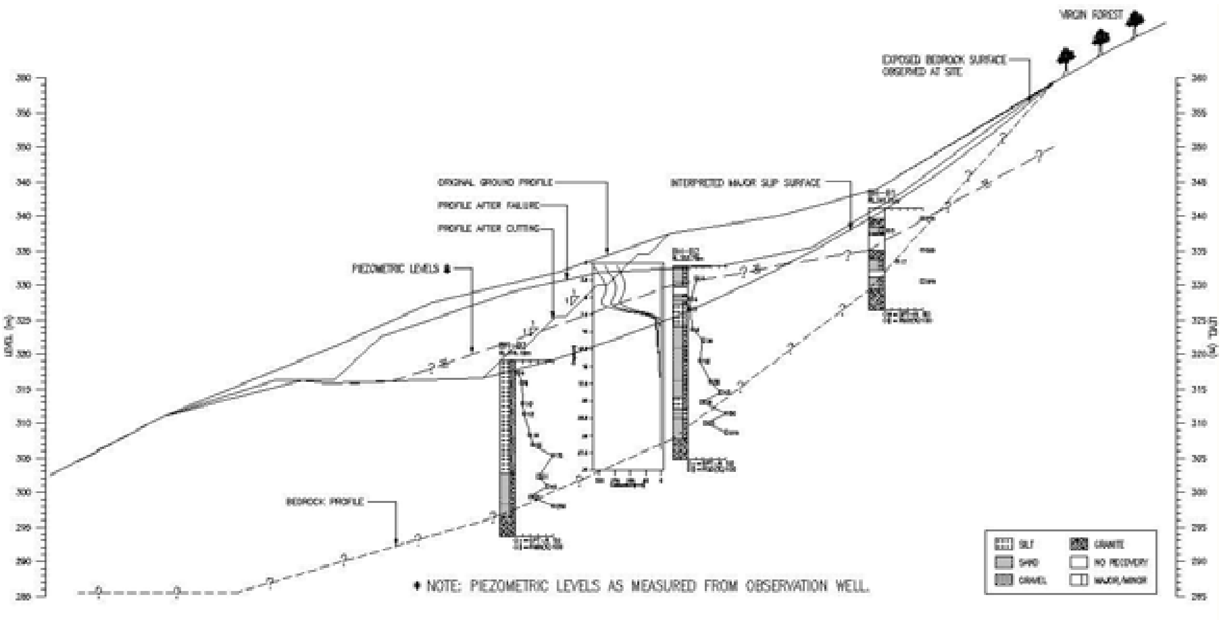

Fig. 10. Interpreted Slip Surface and Piezometric Level

It would be noteworthy to mention that geological structure discontinuities seemed to be involved in the localized failure event. Based on the photos provided, the authors deduced that the cracks on the slope face correspond to the fault as discussed in Section 4.1. One of the photos is shown in Figure 11. Unfortunately, the authors were only involved in the investigation works after the major failure and no further information on the localized failure was available to warrant a detailed study.

Fig. 11. Localised Failure of Slope

5.1 *Topography and Geological Conditions*

The failed slope involved three berms with a total height of 21m. The slope toe was retained by a two meter height gabion wall, followed by a 6m (1V:1.6H), 5m (1V:2.4H) and 10m (1V:1H) height slope respectively. On top of the fill slopes is the platform where the pipeline was laid. There were another three slopes sitting on top of the platform where two of the slopes were also covered with gabion mattress. All the three filled slopes and another two slopes on the platform collapsed following a series of heavy downpour. Figure 12 shows the extent of the failure.

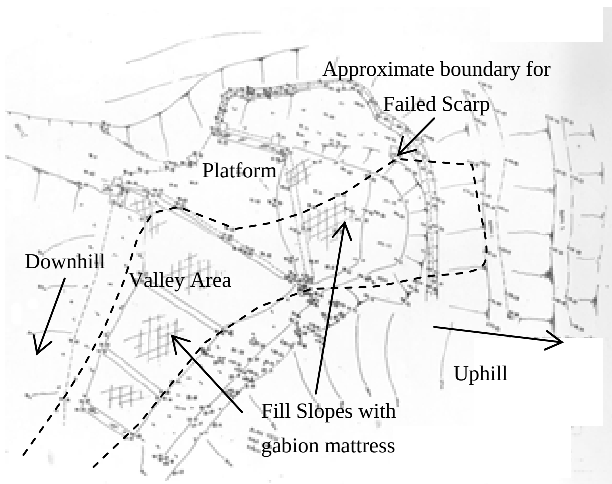

Fig. 12. Plan showing Fill Slope and Boundary of Failed Scarp

The observation at the site after the collapse indicates that the platform was saturated with water. Debris, tree trunks and vegetation on the platform indicated that surface runoff had overflowed the platform and traveled downslope to the lower portion of the valley. The debris and failed mass had traveled more than 120m downhill along the valley as shown in Figure 13. This indicates the high mobility of the collapsed debris, which could be in the form of mud or debris flow. Bedrock was observed at certain exposed surfaces after the collapse, indicating the bedrock level is shallow and the failure resembles a slide along the bedrock surface.

Fig. 13. Travel of Debris and Failed Mass Downhill

The site geology reveals predominantly meta-sedimentary bedrock comprising of interbedded sandstone and siltstone with extensive quartz bands (Geological Survey Malaysia).

5.2 *Subsurface Investigation and Back-Analysis*

Subsurface investigation including of three boreholes were carried out. The interpreted peak shear strength parameters from the C.I.U tests are c_P'=2kPa, ϕ_p'=32^o. Back-analysis was performed using PC-Stabl to estimate the mobilized shear strength parameters during failure. It was revealed that when groundwater level rises near to the ground surface, the mobilized shear strength is closed to the interpreted C.I.U results with the Factor of Safety approximately 1.0.

5.3 *Remedial Works*

In the remedial design, fill embankment was adopted. The fill embankment comprises of rock toe, seven berms of slopes with 5m berm height and a gradient of 1V:2H. A 1.5m width berm with berm drain is provided at every 5m interval.

Fig. 14. Completed View of Filled Embankment

As the fill embankment is built on the previous valley, groundwater tends to flow towards the valley, particularly perching on the bedrock surface. Extensive subsoil drainage such as French drain and drainage blanket had been incorporated in the fill embankment. The French drain was installed in the trench at the center of the embankment footprint prior to filling and serves as the main subsoil drainage path for the embankment. Drainage blankets were installed in the fill slopes and are discharged to the berm drains. Figure 14 shows the completed view of the fill embankment.

6.0 *Conclusions*

Based on the four case histories of slope failure events presented, the findings are summarized as below:

a. Subsurface investigation is an important tool to obtain necessary subsoil information and to established a much better educated guess on the hydro-geological, geological and finally geotechnical models for the design.

b. Instrumentation has played vital role in the failure investigation, in which a slip surface and groundwater regime can be accurately established by inclinometer and piezometer respectively.

c. In additional to the conventional limit equilibrium method, finite element or finite difference methods are strongly recommended to be carried out for the assessment of potential failure mechanism, such as progressive failure.

d. The building up of pore water pressure in relation to rainfall over certain period is still not well understood. This area needs more research works and practical design approaches for slope design. However, the case histories seem to suggest that slope failure events are highly related to prolonged and heavy rainfall.

Among the four case histories of slope failure events presented, three cases are of cut slope failure, which suggest that cut slope has a high frequency of failure. This is probably due to the many uncertainties in identifying and establishing the weak structure, subsoil variation and adverse groundwater level. The authors hoped that these case histories would serve as a lesson to the geotechnical community, that careful planning, proper design and frequent review works are essential for the design of all cut and fill slopes to mitigate the risk of failure.

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