Geotechnical Investigation and Monitoring Results of a Landslide Failure at Southern Peninsular Malaysia (Part 2: Back Analyses of Shear Strength and Remedial Works)

Liew, S. S., Gue, S. S., Liong, C. H.
Director, Gue & Partners Sdn Bhd, Malaysia.
Managing Director, Gue & Partners Sdn Bhd, Malaysia.
Geotechnical Engineer, Gue & Partners Sdn Bhd, Malaysia.

Synopsis

Following the geotechnical investigation and monitoring results, back analyses have been carried out using finite element program and conventional limit equilibrium stability program to establish the mobilised strength of the slope and determine the mechanism of failure. Back-analyses of the soil strength parameters have indicated that post failure shear strength is close to the lower bound of the residual shear strength as interpreted from laboratory results whereas the pre-failure shear strength is lower than the critical state strength. The remedial works were developed using both ϕ1200mm and ϕ1500mm contiguous bored pile (C.B.P.) wall and regrading of the retained slope profile. Extensive subsoil drainage behind the CBP wall and within the failed soil mass has also been installed to release the groundwater pressure. Instrumentation scheme at the CBP wall structure have also been implemented to gather performance results of the wall during construction and post-construction stages. The instrumentation reveals, in normal weather, continuous creep movement during the construction of CBP wall. More rapid creep movement was observed during heavy rain. The creep movement was stabilised after the construction of counter-weight berm and trimming of slope profile.

Keywords

back-analysis, peak strength, critical state strength, residual strength, contiguous bored pile wall, drainage.

1. Back-Analyses for the Collapsed Slope

1.1 Back Analyses Using Conventional Limit Equilibrium Method

The back-analyses were performed using Bishop Circular Method in a computer program (Harald, 1989) developed by Purdue University. The well-defined slip surface and groundwater table as established in the field investigation (refer to Part 1 of this paper) 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 mobilised shear strength along the aforementioned slip surface, the shear parameters, c' and ϕ' are adjusted till the factor of safety is unity (1.0) as a prerequisite for failure in a limit equilibrium analytical model. For Condition (a), the back-analysed mobilised strength parameters on the slip surface are : c' = 0 kPa, ϕ'=24^o. The back-analysed mobilised strength parameters on the slip surface for Condition (b) are : c' = 0 kPa, ϕ'=14.4^o. These analyses are relevant to the shear strength at respective shear strains. For the slope with the original slope geometry to fail, the mobilised shear strength shall overcome the soil strength at the slip surface. However, after the slope collapsed to more stable slope geometry, the creep movement of significant magnitude would justify reduction of the shear strength to, probably, residual strength of the slope material.

A similar back analysis was carried out based on the abovementioned two Conditions with the back-analysed soil strength to search for the most critical slip surface. It was found that the most critical slip surface is somehow shallower than the one determined by the inclinometers. This implies that there must be a relatively weak layer existed along the identified slip surface to precede the most critical slip surface.

1.2 Back Analyses Using Finite Element Method

Back-analyses were also carried out using finite element programme (PLAXIS) with simple Mohr-Coulomb model. Firstly, the original slope profile after cutting before failure (which is the Condition (a) as mentioned in Section 1.1) was modelled in PLAXIS. The shear strength of the predetermined soil zone to model the identified slip surface was then gradually reduced until Factor of Safety of slope was near to unity (1) using the Phi-C Reduction procedure (Brinkgreve, 2002). It was found that the shear strength parameters for the slip surface are : c' = 0.5 kPa, ϕ'=25.9^o when the factor of safety is reduced to 1.04. The back-analysed shear strength is lower than the interpreted laboratory critical state strength parameters which are : c' = 3.0 kPa, ϕ'=29^o. In a detailed review of the shear stress along the developed slip surface, it is noticed that the mobilised shear stress is highly non-uniform. This implies different level of mobilised shear strength along the slip surface. In this case study, the average mobilised shear strength is lower than laboratory peak strength, even slightly lower than the laboratory critical state strength. This is a common phenomenon called progressive failure.

The slope profile after failure was also modelled using the same process as mentioned above. The back analysed shear strength parameters are : c' = 0.5 kPa, ϕ'=15^o with the factor of safety of 1.03.

1.3 Comparison of STED and PLAXIS

In general, the back-analysed strength parameters in both methods are fairly similar, except the friction angle of PLAXIS in Condition (a) is about 2^o higher than that of PC-STABL6. Table 1 summarises the back-analysed results.

It is expected that the back-analysed shear strength in Conditions (a) and (b) correspond to critical state strength and residual strength respectively.

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 (PCSTABL)	0	24^o	0	14.4^o
2. Back Analysis (PLAXIS)	0.5	25.9^o	0.5	15^o

2. Residual Strength and Peak Strength

2.1 Correlations of Residual Strengths

Skempton (1964), Mesri and Cepeda-Diaz (1986) presented some correlations between residual friction angle and clay size fraction, liquid limit or clay mineralogy. The residual friction angle of soil decreases with the increase in the percentage of clay size fraction and liquid limit.

In this investigation, the liquid limit of the samples generally range from 38 to 52 (refer to Table 2 in Part 1). In Figure 1, the corresponding residual friction angle ranging from 26^o down to 18^o is obtained for the abovementioned liquid limit.

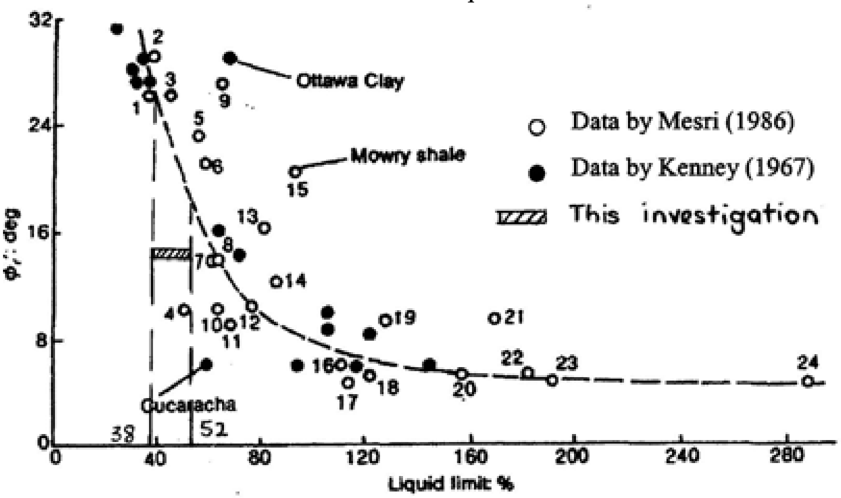

Figure 1 Residual Friction Angle VS Liquid Limit (Mesri and Cepeda-Diaz)

The percentage of clay size fraction for collected specimen generally varies from 13% to 20%. Figure 2 shows the corresponding residual friction angle of 29^o to 24^o. This is close to the results obtained from the residual friction angle-liquid limit relationship. Table 2 summarises the ranges of residual strength obtained from different methodology.

2.3 Discussions on Peak and Residual Strength

Despite C.I.U. test results are very consistent, this does not imply that the C.I.U. test shall be used for the remedial design of a failed cut slope. If the C.I.U. shear strength parameters (c'=3.5 kPa, $\phi'=32^o$) are applied in the slope analysis based on the identified slip surface for the slope profile before failure, then the slope would not fail. Factor of safety of 1.49 is yielded for the above slope analysis (using Bishop circular method), even when the groundwater table is close to the ground surface. Therefore, it is credible to deduce that there is an existence of thin layer at the slip surface of exceptionally low shear strength than the corresponding shear strength in both Condition (a) and (b). This slip surface is difficult to be determined accurately in the subsurface investigation unless with the help of inclinometer results. Such thin weak layer is believed to have experienced substantial shearing strain prior to the incidence of failure and therefore exhibits the average mobilised shear strength lower than critical state strength in Condition (a) and lower bound of to residual strength in Condition (b).

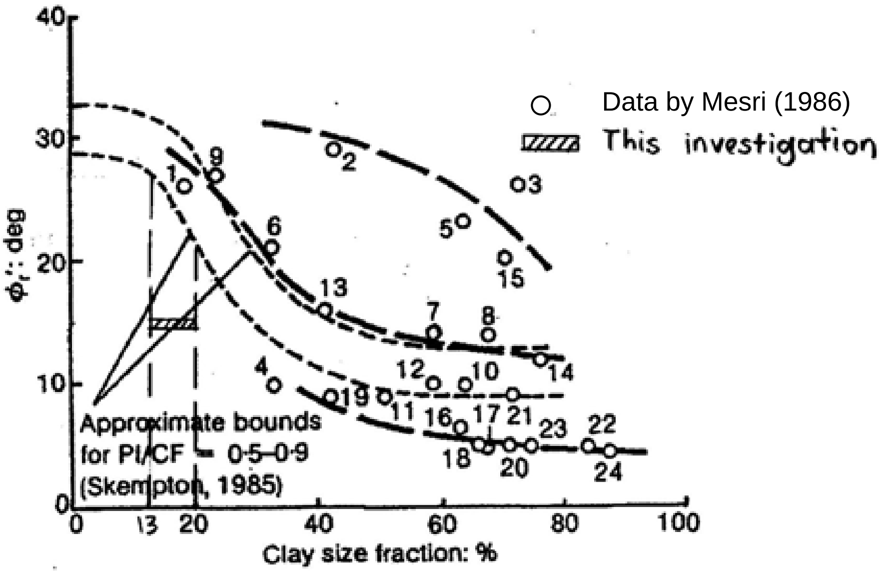

Figure 2 Residual Friction Angle VS Clay Size Fraction (Mesri and Cepeda-Diaz)

Based on the above discussions, it can be concluded that the shear strength on the critical slip surface must be sufficiently low for the original cut slope to collapse. Therefore, the shear strength to be used for the remedial design should be close to the residual strength.

Table 2 Comparison between Residual Strength and Peak Strength for Correlations and Laboratory Results

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°	0	18°	-	-	-	-
2. Clay Size Fraction	0	27°	0	21°	-	-	-	-
3. Reversal Shear Box Tests on Reconstituted Samples	31.4	21°	0	14°	39	30°	5.9	21°
4. C.I.U. on Undisturbed Samples	Peak Strength Parameters : c'_p = 3.5kPa, ϕ'_p = 32 °							
	Critical State Strength Parameters : c'_{cr} = 3.0kPa, ϕ'_{cr} = 29 °							

3. Remedial Works for Collapsed Slope

As the proposed building has to be constructed within the upper portion of the collapsed slope, strengthening works on the collapsed slope is required to ensure stability of building platform. Various types of retaining structures, such as reinforced soil wall, reinforced concrete wall and contiguous bored pile (CBP) wall have been explored.

It was found that there are a few disadvantages for the first two options. Firstly, the bearing capacity of the ground is poor and the retaining wall must be supported by pile foundation. The construction of the retaining wall also requires temporary excavation of the collapsed slope to provide temporary working space. This temporary excavation would likely to trigger another slope collapse, as the ground creep movement has been shown to be active as shown in the instrumentation results. Therefore, the contiguous bored pile (CBP) wall was adopted to minimize construction disturbance to the ground. The CBP wall location is shown in Figure 5 (Part 1 of this paper).

As groundwater seepage is very active at the collapse slope, effective drainage blanket behind the CBP wall and subsoil drainage in the remedial slope were designed and constructed to drain out and lower groundwater pressure to further improve the stability. From the observed discharged seepage water, it is confirmed that the provision of these drainage system has efficiently control the groundwater profile.

4. Instrumentation for CBP Wall

Instrumentations such as inclinometers and strain gauges were installed for the two (2) CBP piles, namely A17 and B4.

The inclinometers registered unexpected large movement when the first readings (taken on 22 May 2002) were compared to the initial readings. (Refer to Figure 8 of Part 1). Maximum pile head movements of 92mm and 79mm were observed at IN-A17 and IN-B4 respectively. The alarm was raised that the slope may be unstable as the movement was significant. Decision was made to immediately construct counter-weight berm in front of the CBP wall to stabilise the slope.

The second inclinometer readings (taken on 29 May 2002) revealed that there was additional pile head movement of 10mm and 13mm for IN-A17 and IN-B4 respectively. It was concerned that the CBP piles could be structurally damaged due to large lateral movement experienced. Pile Integrity Test (PIT) was later carried out to assess the structural integrity of the bored piles. The test results found that no abnormalities were encountered within the pile length.

Subsequent inclinometer readings (July - August) showed much lower rate of wall movement. A sudden increased of wall movement was found within the first two weeks of September 2002. Inclinometer IN-A17 and IN-B4 recorded additional pile head movement of 245mm and 255mm within the period. Flexural cracks on exposed side of the CBP capping beam was observed on 10 September 2002. The temporary drainage at site was not satisfactory as water was found ponding behind the CBP wall. The platform behind the CBP wall was rather high while the counter-weight berm was yet to be formed to the final profile. In addition to the above, heavy and continuous rainfall at the end of August and beginning of September is the major triggering factor of the slope movement. This can be observed in Figure 8, where a heavy rainfall of 79.2mm was recorded for 21 August 2002.

Immediate actions such as diverting temporary drainage, trimming of the upper platform and speeding up of the construction of counter-weight berm were taken. The movement of the CBP wall and slope were then stabilised.

The third inclinometer, IN-C was installed at 5.5m behind the CBP wall on October 2002 to further assess the movement of the slope. The average ground movement rate recorded for IN-C is less than 1mm/day. However, continuous creep movement was observed. For a period of three (3) months after installation, IN-C has registered a cumulative maximum movement of 24mm. The subsequent readings thereafter indicated that the landslide has been stabilised after the designed slope profile was completed.

5. Conclusions and Recommendations

The following findings can be summarised from the back-analyses:

a. The back-calculated strength for the two-berm cut slope is lower than the critical state strength interpreted from C.I.U. tests.
b. The back-calculated strength for the failed slope is close to the lower bound of residual strength interpreted from M.R.D.S.B. tests.

The following recommendations are proposed for the slope design and remedial works of failed slope:

a. For slopes not subject to previous failure, soil strength of not higher than critical state strength derived from C.I.U. tests shall be used in stability analysis to avoid long-term delayed failure or progressive failure.
b. For slopes experienced previous instability, residual strength derived from M.R.D.S.B. tests or preferably ring shear tests shall be used for remedial design.
c. For slope with high groundwater, effective drainage system, such as surface drains and subsoil drains, shall be implemented to improve slope stability.
d. Prevailing to the interpreted laboratory residual strength, back-analysed shear strength along the identified slip surface shall be used in remedial design.
e. For cut slope design, site inspection and instrumentation shall be carried out to gather evidence of creeping movements and the shear strain in the soil mass, so that appropriate designed shear strength can be selected.

REFERENCES

[1] Brinkgreve, R.B.J. 2002 Plaxis-Finite Element Code for Soil and Rock Analyses Version 8, A.A. Balkema.
[2] Mesri, G & Cepeda-Diaz, A.F. 1986. Residual Shear Strength of Clays and Shales, *Geotechnique* 36, No.2, 269-274.
[3] Harald, W. Van Aller, P.E., 1989. Sted Version 6.5 The Smart Editor for Pcstabl.
[4] Skempton, A.W. 1964. Long-Term Stability of Clay Slopes, *Geotechnique* 14, No. 2, 75-101.