Geotechnical Investigation and Monitoring Results of a Landslide Failure at Southern Peninsular Malaysia (Part 1: Investigating Causes of Failure)

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

This paper presents a case study of a well documented landslide at hilly terrain with colluvium deposits. Investigation works had been carried out using exploratory boreholes, laboratory tests, and various geotechnical instruments to measure the landslide movements and groundwater variation. The use of inclinometers has successfully detected multiple slip surfaces within the failed masses and also indicated the on-going active landslide creep movements. Numbers of laboratory tests, such as Consolidated Isotropically Undrained (C.I.U.) Triaxial Shear Tests with Pore Pressure Measurements for peak shear strength, Multiple Reversal Direct Shear Box (M.R.D.S.B.) Tests on reconstituted soil samples for residual shear strength, Unconsolidated Undrained (U.U.) Triaxial Shear Tests for undrained shear strength, clay mineralogy and petrography for identification of landslide masses derivatives, and the meteorological records have been carried out and compiled to investigate the causes of failure.

Keywords

landslide, instrumentations, slip surface, multiple reversal shear box test, rainfall record.

1. Introduction

A proposed building was constructed on a cut platform. After cutting of a two-berm slope (gradient 1V:1.5H) for the building platform, the cut slope collapsed following a heavy downpour. On top of the failed cut slope, there was another proposed structure yet to be constructed. A comprehensive geotechnical investigation was carried out to investigate the causes of the failure and to propose remedial measures.

2. 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 regional geological map of Malaysia (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 shown in Figure 1. It was observed that the vicinity of the site comprises of different lithological units.

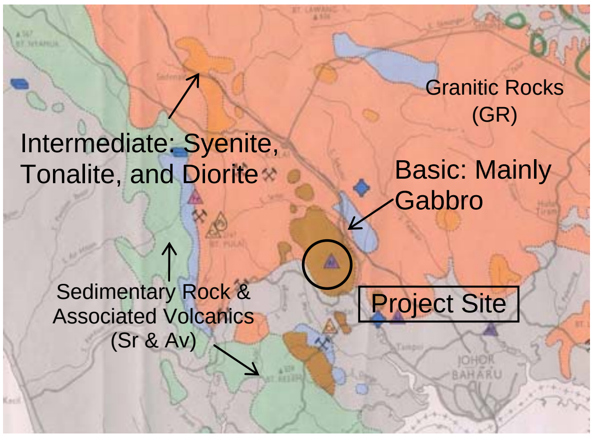

Figure 1 Regional Site Geology and Mineral Resources Map, 1982

Figure 2 Front View of Failed Slope

3. Site Conditions and Observations

A site inspection was carried out shortly after the slope collapsed. Whitish silt material was found on the cut surface of the failed slope as shown in Figure 2. Tension cracks were also observed at top of slope as shown in Figure 3.

Figure 3 Tension Cracks at Top of Slope

Figure 4 Water Seepage at Berm

Figure 4 shows water seepage at various locations, indicating potential high groundwater level at the failed slope. Small boulders were also observed on the surrounding of the slope. The existence of boulders (diorite and gabbro) within the subsoil was further confirmed during the borehole exploration.

4. Subsurface Investigation and Instrumentation Works

Subsurface investigation and instrumentation programmes consisting of ten boreholes, three inclinometers, six observation wells and one standpipe piezometer were planned and implemented to investigate the causes of failure, to propose remedial measures and for geotechnical design of the upper proposed building. The layout of the boreholes and instrumentations is shown in Figure 5. Three boreholes, namely BH-1, BH-2 and BH-3, were sunk within the failed mass. Upon completion of boring operation and sample collection, inclinometers, IN-1, IN-2 and IN-3 were installed in the boreholes. Apart from the inclinometers, three observation wells, OW-1, OW-2 and OW-3 were also installed at 1m away from boreholes BH-1, BH-2 and BH-3 respectively. Liew & Gue (2001) have presented a similar investigation for post glacial deposit in East Malaysia, in which the inclinometers have successfully determined the creep movements and slip surface. Figure 6 shows the interpreted subsoil profiles. The overburden material is generally weak, with SPT-N ranging from 0 to 15.

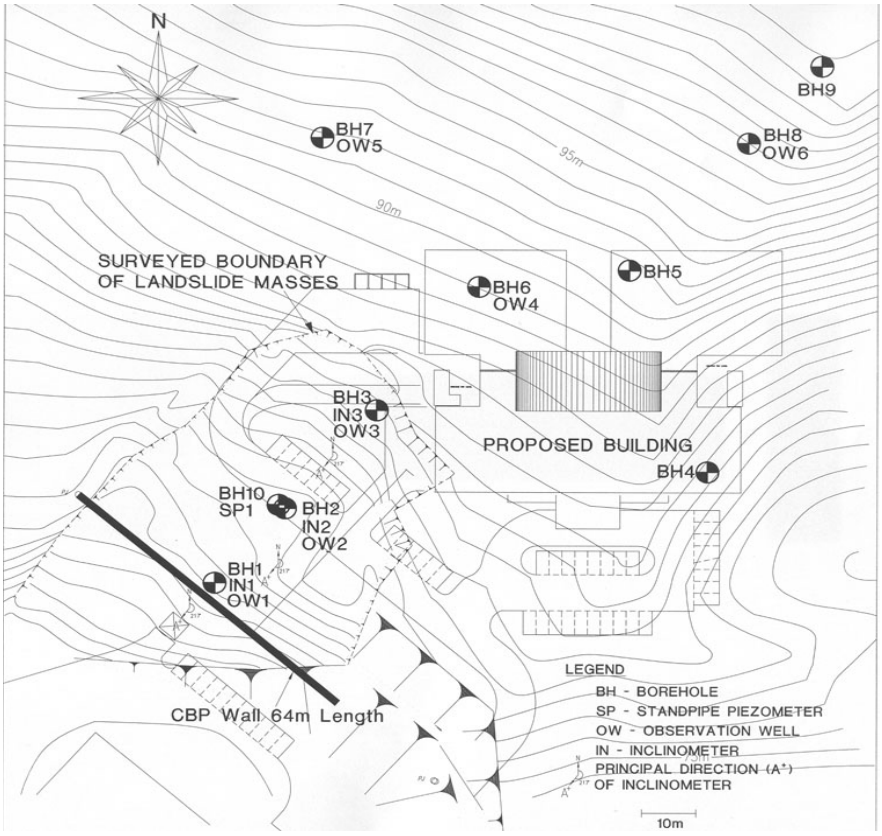

Figure 5 Layout of Borehole and Instrumentation Plan

When the inclinometers detected a slip surface within the failed slope, an additional borehole, BH-10 was sunk 1m away from BH-2 to collect undisturbed samples near the identified slip surface for laboratory strength tests. A standpipe piezometer, SP-1 was also installed in borehole, BH-10 at 11.0m below ground surface, where the slip surface was identified.

Inclinometers, IN-1 and IN-2, were sheared off at 10.5m and 12.0m below ground level shortly after installation. Subsequent monitoring revealed that inclinometer, IN-2, was sheared off again at another higher location, 6.0m below ground level. Finally, IN-3 was sheared off at 2.5m below ground level. The first major slip surface was identified when the three inclinometers were sequentially sheared off. The three shear-off points of the inclinometers resemble a well defined circular slip surface when joined together. The circular slip surface also agrees well with the tension cracks and bulging of the slope toe indicating where the slip surface starts and ends on the slope profile. The second shear off point at inclinometer, IN-2, revealed another minor slip surface formed after the first major slip surface. The inclinometer results are shown in Figure 7, which shows the interpreted multiple slip surfaces in the failed slope. Figure 7 indicates that collapsed mass has resultant movements towards south-west direction.

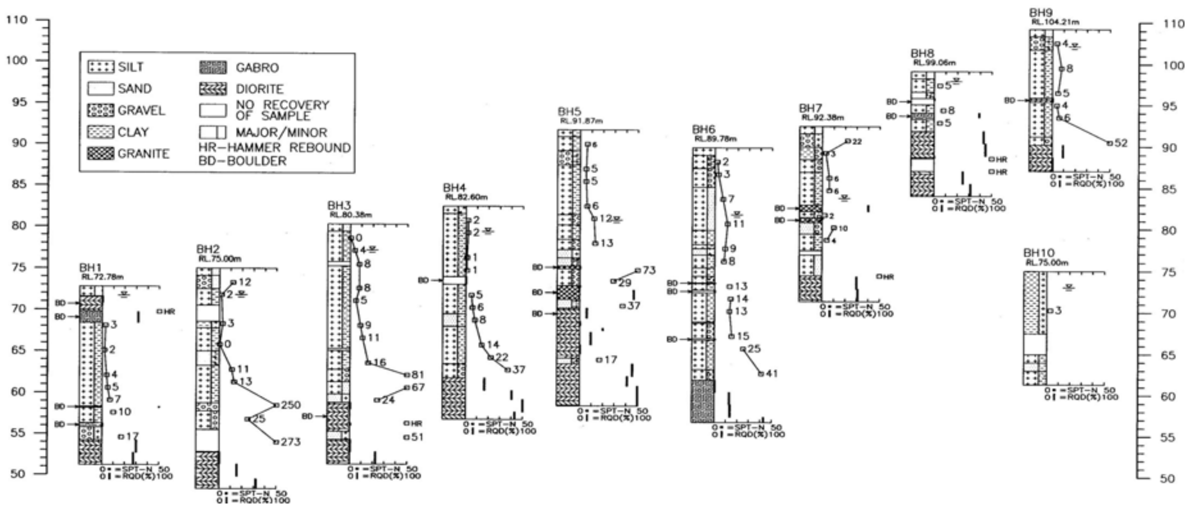

Figure 6 Interpreted Subsoil Profile

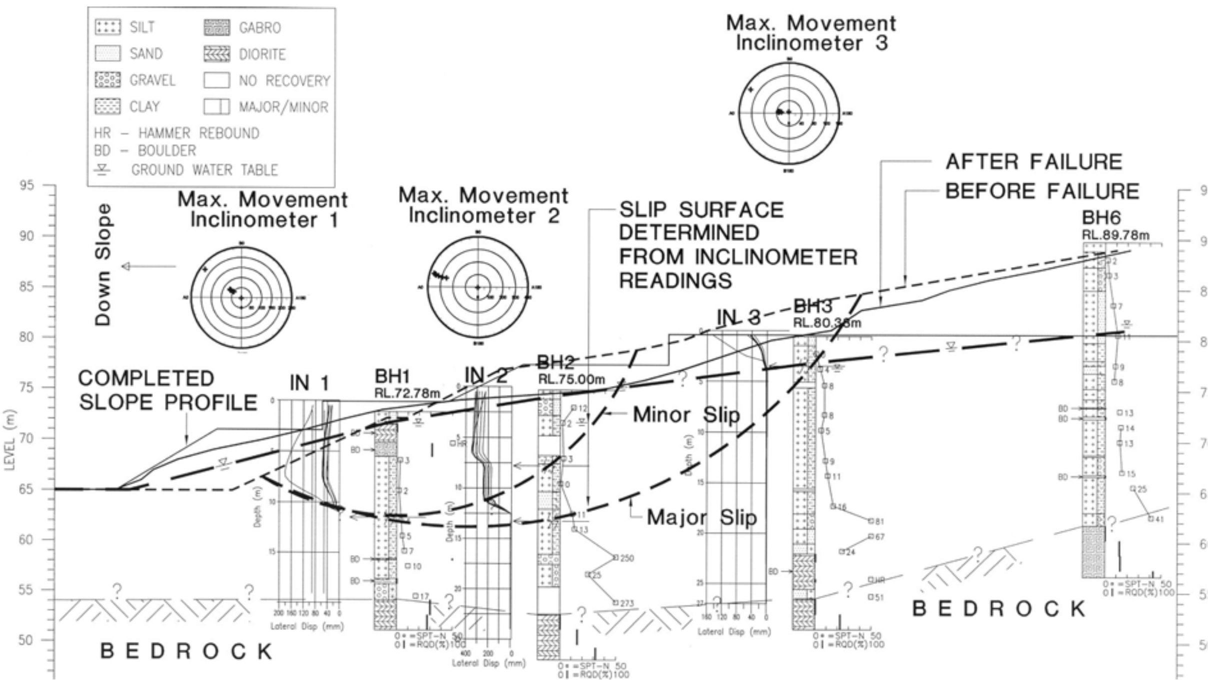

Figure 7 Multiple Slip Surfaces Interpreted from Inclinometers

The rates of maximum movement of the inclinometers are shown in Figure 8 together with the daily and cumulative rainfall records. Inclinometer, IN-2, registered the largest movement rate during the initial monitoring period as it was installed in the middle of the collapsed mass. The trend of movement rate for the other inclinometers was very similar and consistent. The movement rate started with a peak and reduced gradually. However, during investigation, the movement rate increased before the inclinometers were damaged. It is observed that the increased movement rate corresponds with an extremely heavy rain recorded on 27 Dec 2001.

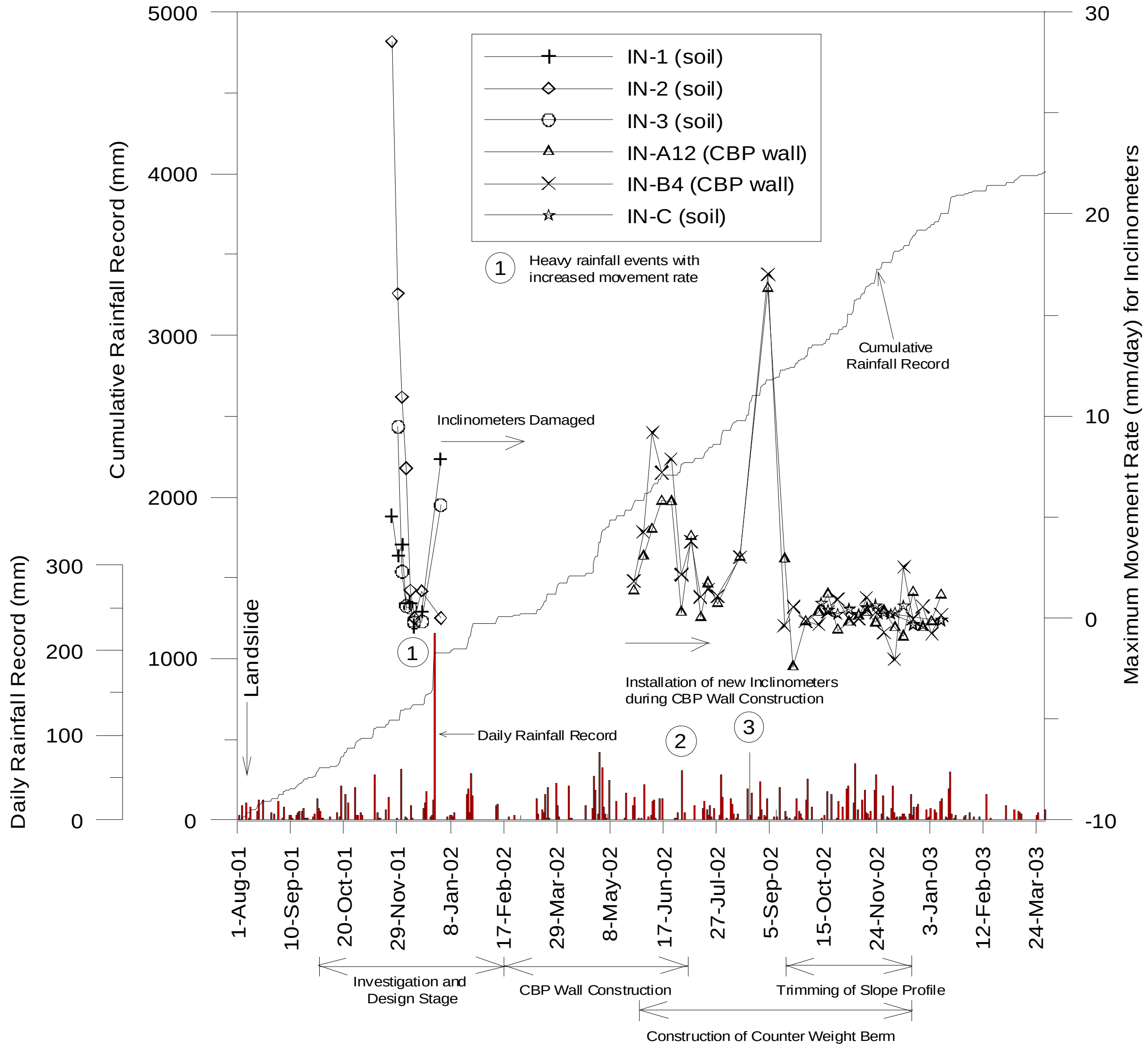

Figure 8 Rainfall Record and Maximum Movement Rate for Inclinometers

The groundwater table was established from the six observation wells and one standpipe piezometer to obtain accurate groundwater levels. Extra precaution, such as water bailing in these groundwater instruments, has been carried out for re-establishing of equilibrium of water level during the period of investigation. The monitored groundwater table within the failed slope was high, ranging from the ground surface to 2.9m below ground level as tabulated in Table 1.

Apart for groundwater level measurement, the observation wells and standpipe piezometer were also coincidently used to locate the slip surface. A dipmeter was lowered into the observation wells and standpipe piezometer. The maximum reach of the dipmeter in the tubing was recorded in each monitoring. The instrument tubing is most likely sheared off after excessive post-failure creep movement and resulting in blockage to the dipmeter to reach the full depth of the tubing.

The slip surface interpreted from the blockage of the observation wells, OW-1 and OW-3, corresponds well to the slip surface detected in the adjacent inclinometers. The slip surface located by observation well, OW-2 and piezometer, SP-1 is most likely to be the minor slip surface, which was also detected in the inclinometer IN-2.

Table 1 Groundwater Level and Maximum Depth reached by Dipmeter after Blockage

Instrument No	Water Level Below Ground Level (m)			Maximum Depth of Dipmeter Reached After Blockage (m)
	Highest	Average	Lowest	
OW-1*	0.76	0.80 - 0.90	1.70	10.98
OW-2*	0.00	0.05	0.50	8.28
OW-3*	2.63	2.70	2.90	3.03
OW-4	8.25	8.80	9.18	-
OW-5	8.08	8.80 - 8.90	9.00	-
OW-6	10.99	11.00	11.02	-
SP-1*	-0.02	0.15	0.20	7.18

Note: * Instruments at the landslide area.

4.1 Laboratory Test Results

A series of the following laboratory tests were carried out on the samples obtained from the subsurface investigation works:

1. Atterberg limits,
2. Particle size distribution,
3. Multiple reversal direct shear box test,
4. Unconsolidated undrained triaxial test,
5. Consolidated isotropically undrained triaxial test with pore pressure measurement,
6. Unconfined compressive strength test on rock,
7. X-ray diffraction test,
8. Petrographic analysis.

Selected test results near to the slip surface are presented in the following sections.

4.1.1 Atterberg Limits and Particle Size Distribution

Based on the British Soil Classification System, most of the soil samples collected near the slip surface is clayey silt of intermediate to high plasticity as summarised in Table 2.

Table 2 Atterberg Limits and Particle Size Distribution

Borehole	Depth (m)	Liquid Limit	Plastic Limit	Clay (%)	Silt (%)	Sand (%)	Gravel (%)
BH-1 (P4/D4)	10.5 – 10.95	48	33	16	58	23	3
BH-1 (P6/D6)	13.5 – 13.95	43	28	10	60	24	6
BH-2 (P5/D6)	12.0 – 12.45	52	35	16	72	12	0
BH-2 (P6/D7)	13.5 – 13.95	42	31	13	67	19	1
BH-3 (P1/D2)	1.5 – 1.95	65	44	20	76	4	0
BH-10 (UD2)	11.0 – 11.95	38	23	2	8	90	0
BH-10 (UD3)	12.0 – 12.60	49	33	15	55	26	4

4.1.2 Consolidated Isotropically Undrained (C.I.U.) Triaxial Test

Eleven (11) numbers of C.I.U. tests were carried out on the thin wall and Mazier samples. Figures 9 and 10 show both the T-S plot for the interpreted peak strength (c' = 3.5 kPa and

ϕ'=32^o) and critical state strength (c' = 3.0 kPa and ϕ'=29^o). As shown in the graphs, the C.I.U. results are fairly consistent.

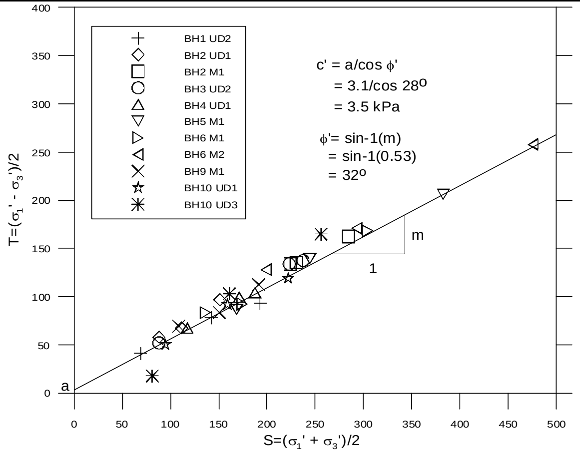

Figure 9 T-S Plot (Peak Strength) for CIU Tests

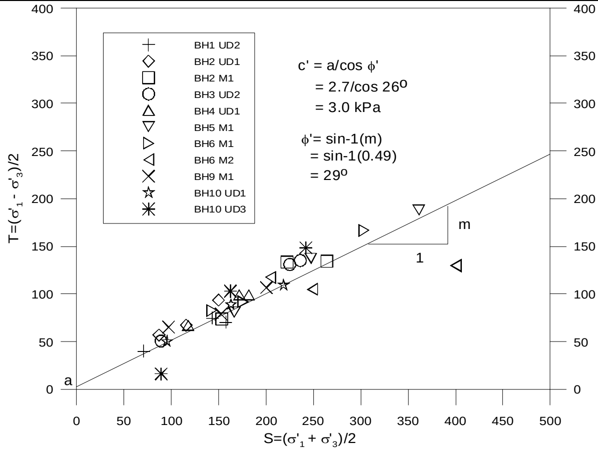

Figure 10 T-S Plot (Critical State Strength) for CIU Tests

4.1.3 Multiple Reversal Shear Box Test

Ten (10) numbers of multiple reversal direct shear box tests were carried out on the reconstituted samples collected from the boreholes and failed mass. The main objective of the reversal shear box test is to obtain the residual strength of the soil. It is particularly relevant in designing a remedial work for a failed slope with identified shear surface and to explore its relationship with the active creep movement of the failed soil mass. In the reversal shear box test, slickensided surface is artificially formed after significant re-shearing of the sample. The strength of the shearing soil is expected to be reduced to residual value when well-defined slickensided surface is fully developed. This is due to the rearrangement of the soil particle along the shear surface into a smoother surface and minimising the interlocking effect of the soil particles. During the rapid multi-reversal, the reversal test shows gradual reduction of shear stress in each shearing.

The results of the shear box test are plotted in Figures 11 to 12. It can be observed that the results of the shear box tests are fairly scattered.

Figure 11 shows the upper bound of the peak strength of : c'=39.0 kPa, ϕ'=30^o while the lower bound value of : c'=5.9 kPa, ϕ'=21^o. The average shear strength obtained is : c'=15.7 kPa, ϕ'=24^o.

Figure 12 shows the upper bound of the residual strength of : c'=31.4 kPa, ϕ'=21^o while the lower bound value of : c'=0 kPa, ϕ'=14^o. The average shear strength obtained is : c'=5.9 kPa, ϕ'=20^o. The scatter of interpreted residual strength is rather large and could be largely due to inconsistency in generating the smooth shearing surface in this particular soil type during the reversal shearing process. A continuous large strain shearing in one direction, like ring shear test, could have produced a more consistent residual strength.

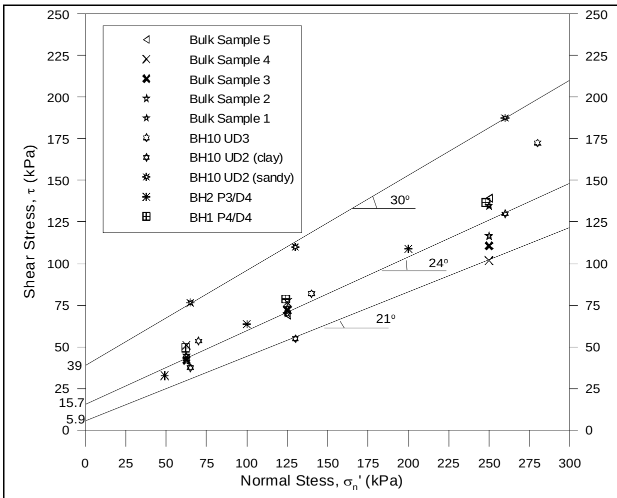

Figure 11 Shear Stress VS Normal Stress Plot (Peak Strength) for Direct Shear Box Test

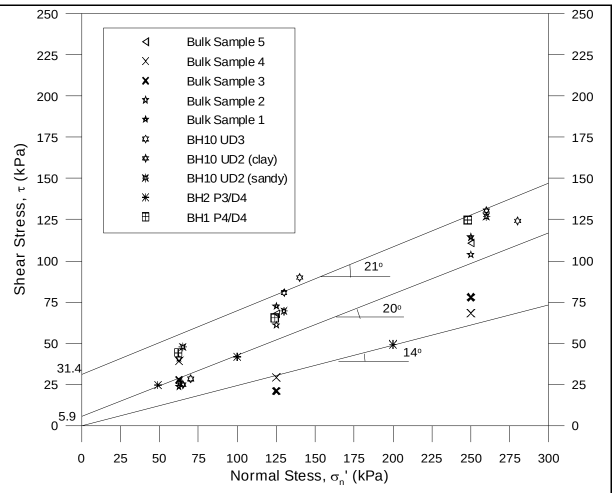

Figure 12 Shear Stress VS Normal Stress Plot (Residual Strength) for Multiple Reversal Direct Shear Box Test

4.1.4 X-ray Diffraction (XRD) and Petrographic Examination

In order to confirm the rock type and its derivative, four (4) rock samples were collected at the site for XRD and petrographic tests. XRD analysis was performed for three (3) samples as the samples were too weathered and thin section cannot be prepared. Petrograhic examination was carried out for the fourth rock sample, which was collected from the borehole. The results are summarised in Table 3.

Table 3 Locations and Types of Rock Samples

Sample	Location	Colour	Rock Name
1	Failed Scarp Area	White	Weathered granite
2	Proposed Building Footprint (Upper platform)	Brown (result of oxidation)	Weathered medium to fine grained gabbro
3	Failed Scarp Area	White to light Brown	Weathered gabbro
4	BH-8	Dark grey	Medium grained olivine gabbro

5. Summary

From the instrumentation results and a series of laboratory tests, the following findings can be summarised:

a. It is difficult to detect the pre-existed slip surface for a slope unless there are evidences of tension cracks, observable surface movements and measured subsoil deformation by inclinometers indicating the slow creep movement of an unstable slope.
b. Groundwater regime is very sensitive in slope stability. Therefore the groundwater regime shall be credibly established for engineering assessment.
c. The use of M.R.D.S.B. test may not necessarily produce the residual strength as the generation of smooth surface is not in a continuous direction for large shearing strain. Alternatively, ring shear test can be considered to obtain more realistic residual strength.

The following recommendations are proposed for the determination of earlier evidence of slope instability:

a. Trenches can be dug to expose the slickensided surface for shallow slip surface and collect samples for strength test.
b. To detect slope creep movement and determine slip surface, inclinometers are highly suggested at the slope with frequent monitoring during subsurface investigation, preferably at the slope toe, followed by the mid slope and finally the slope crest.
c. Observation wells shall be installed to determine overall groundwater regime and piezometers at specific strata for piezometric level.

REFERENCES

[1] Liew, S.S. & Gue S.S. (2001) Massive Movements of Post Glacial Deposition in Kundasang Areas, GSM-IEM Forum: "Engineering Geology and Geotechnics of Slopes". Malaysia.
[2] Mineral Resources Map of Johor 1982, Geological Survey of Malaysia.