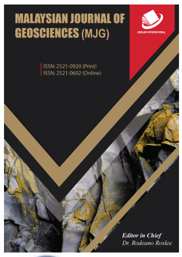
Malaysian Journal of Geosciences (MJG)

**DOI :** https://doi.org/10.26480/mjg.01.2018.09.17

ZIBELINE INTERNATIONAL PUBLISHING

***ISSN: 2521-0920 (Print)***
***ISSN: 2521-0602 (Online)***

***CODEN : MJGAAN***




CrossMark
click for updates

# ENGINEERING GEOLOGICAL ASSESSMENT (EGA) ON SLOPES ALONG THE PENAMPANG TO TAMBUNAN ROAD, SABAH, MALAYSIA

**Rodeano Roslee[1,2]*, Felix Tongkul [1,2]**

***[1] Natural Disaster Research Centre (NDRC), Universiti Malaysia Sabah***
***[2] Faculty of Science and Natural Resources, Universiti Malaysia Sabah***
****Corresponding Author Email: rodeano@ums.edu.my***



**ARTICLE DETAILS**

***Article History:***

Received 12 November 2017
Accepted 12 December 2017
Available online 1 January 2018

**ABSTRACT**

This study focused on the engineering geological investigation of slope failures along Penampang to Tambunan road, approximately 12th km to 101th km from Kota Kinabalu city, Sabah, Malaysia. The area is underlain by the Crocker Formation (Late Eocene to Early Miocene age) and the Quaternary Deposits (Recent age). These rock units show numerous lineaments with complex structural styles developed during several regional Tertiary tectonic activities. The tectonic complexities influenced the physical and mechanical properties of the rocks, resulting in a high degree of weathering and instability. The weathered materials are unstable and may experience sliding due to by high pore pressure and intensively geomorphological processes. In this study, a total of 31 selected critical slope failures were studied and classified into two main groups: rock slope and soil slope. Failures in soil slopes (including embankments) are 21 (67 %) whereas 10 of all failures (33 %) of rock slope. Soil slope failures normally involved large volumes of failed material as compared much rock slopes, where the failures are mostly small. Of the 21 failures in soil slopes, 15 (71 %) are embankment failures making them 48 % of all types of failures. Physical and mechanical properties of 84 soil samples indicated that the failure materials mainly consist of poorly graded to well graded materials of clayey loamy soils, which characterized by low to intermediate plasticity content (9 % to 28 %), containing of inactive to normal clay (0.34 to 1.45), very high to medium degree of swelling (5.63 to 13.85), variable low to high water content (4 % to 22 %), specific gravity ranges from 2.57 to 2.80, low permeability (9.66 X $10^{-3}$ to 4.33 X $10^{-3}$ cm/s), friction angle ($\phi$) ranges from 7.70˚ to 29.20˚ and cohesion (C) ranges from 3.20 KPa to 17.27 KPa. The rock properties of 10 rock samples indicated that the point load strength index and the uniaxial compressive strength range classified as moderately week. Kinematics slope analyses indicates that the variable potential of circular, planar, wedges and toppling failures modes as well as the combination of more than one mode of aforementioned failure. Rock and soil slopes stability analysis indicates that the factor of safety value as unsafe (0.52 to 0.98). Engineering geologic evaluation of the study area indicates that the slope failures took place when rock and soil materials were no longer able to resist the attraction of gravity due to a decrease in shear strength and increase in the shear stresses due to internal and external factors. Internal factors involve some factors change in either physical or chemical properties of the rock or soil such as topographic setting, climate, geologic setting and processes, groundwater condition and engineering characteristics. External factors involve increase of shear stress on slope, which usually involves a form of disturbance that is induced by man includes removal of vegetation cover, induced by vehicles loading and artificial changes or natural phenomenon such as tremors. Development planning has to consider the hazard and environmental management program. This engineering geological study may play a vital role in slope stability assessment to ensure the public safety.

**KEYWORDS**

Engineering Geology, Kinematics Analysis, Slope Stability Analysis, Sabah & Malaysia.

## 1. INTRODUCTION

This paper deals with the engineering geological investigation study of 31 selected critical slopes with the aims of analysis the physical and mechanical properties of soil and rock, calculate the factor of safety for slopes and to evaluate the main factors contributing to slope failures. The study area is located the stretch between 12th km to 101th km from Kota Kinabalu city, Sabah to the town of Tambunan in Sabah, which connecting the lowland areas of the west coast to the interior regions of Sabah, Malaysia. The study area is bounded by longitudes line 116° 15' to 116° 30' E and latitudes line 05° 42' to 05° 55' N (Figures. 1 & 2). The 89 km length study area, crosses over 90 % rugged mountainous terrain with a different of elevation exceeding 1000 m. Part of this highway is constructed across the steep slopes of the Crocker Range, creating problems of slope and stability especially during periods of intense rainfall. Since it's opening in 1980, the problem of slope stability has adversely affected the use of the highway. The Public Work Department of Malaysia (JKR) authority has started a program of repairing and rehabilitation of slope failures since 1990 to improve the highway. This work is still going on today.

## 2. METHODOLOGY

Several classifications can be used to describe slope failures. For this study in the topics, the types of slope failures were classified according to the proposals of a group researcher [1]. In this system, slope failures are classified into two main groups: soil slope failures and rock slope failures. Soil slope failures were divided into slides (T1), slumps (T2), flows (T3), creep (T4) and complex failures (T5) whereas rock slope failures were divided into circular (B1), planar (B2) and wedge failures (B3) together with toppling (B4). In this study, only failures with volume exceeding 10 $m^3$ were considered, since failures involving smaller volume did not generally affect the road users. On the basis, the slope failure was divided into three groups: small (10 – 50 $m^3$), Medium (50 – 500 $m^3$) and Large (> 500 $m^3$). For each slope failures that were studied (Figure 2), type of failures, the geometry of the slope, geological background characteristics, weathering characteristics, ground water condition, discontinuity characteristics, physical and mechanical of the sliding materials and an interpretation of the factors causing the failure based on field observations were recorded. Soil and rock samples from the study area were collected during field mapping for detailed laboratory analysis.

The laboratory works such as classification tests (grain size, atterberg limit, shrinkage limit, specific gravity and water content), permeability test, consolidated isotropically undrained (CIU) test, rock uniaxial compressive strength and point load test were carried out in compliance and accordance to British Standard Code and ISRM [2-6].

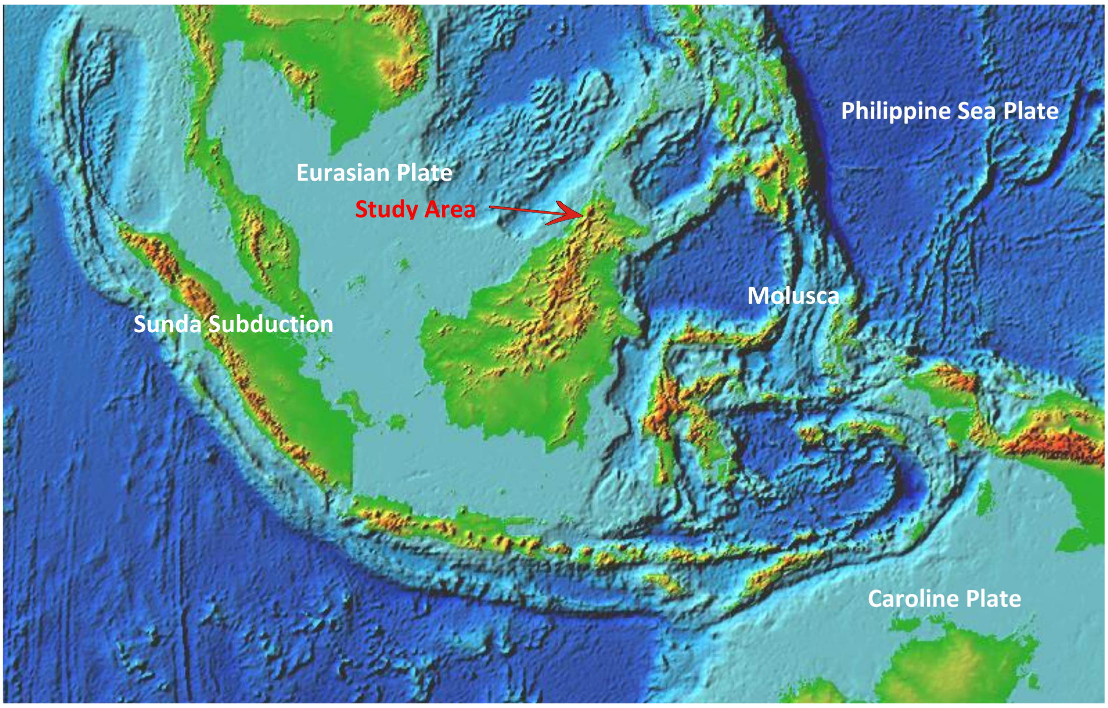



**Figure 1:** Location of the study area [7].

For the soil slopes stability analysis, using the *"SLOPE/W"* software was done successfully to determine susceptibility of the slopes to shallow non-circular slides based on the determination of factor of safety values, which are common in the study area [8]. The advantage of these methods is that in its limit equilibrium calculations, forces and moments on each slice is considered.

Discontinuity orientation data has been collected from ten (10) selected of rock slope failure by random method. For each rock slope failures that were studied, the geometry of the slope, dip direction and dip value, persistence, roughness, unevenness, aperture, infilling material, water condition, weathering, geological background characteristics, engineering properties of the sliding materials and an interpretation of the factors causing the failure were recorded. Determination of discontinuities sets, critical discontinuities plane, potential mode and rock slope stability analysis has been performed by RockPack III program [9].

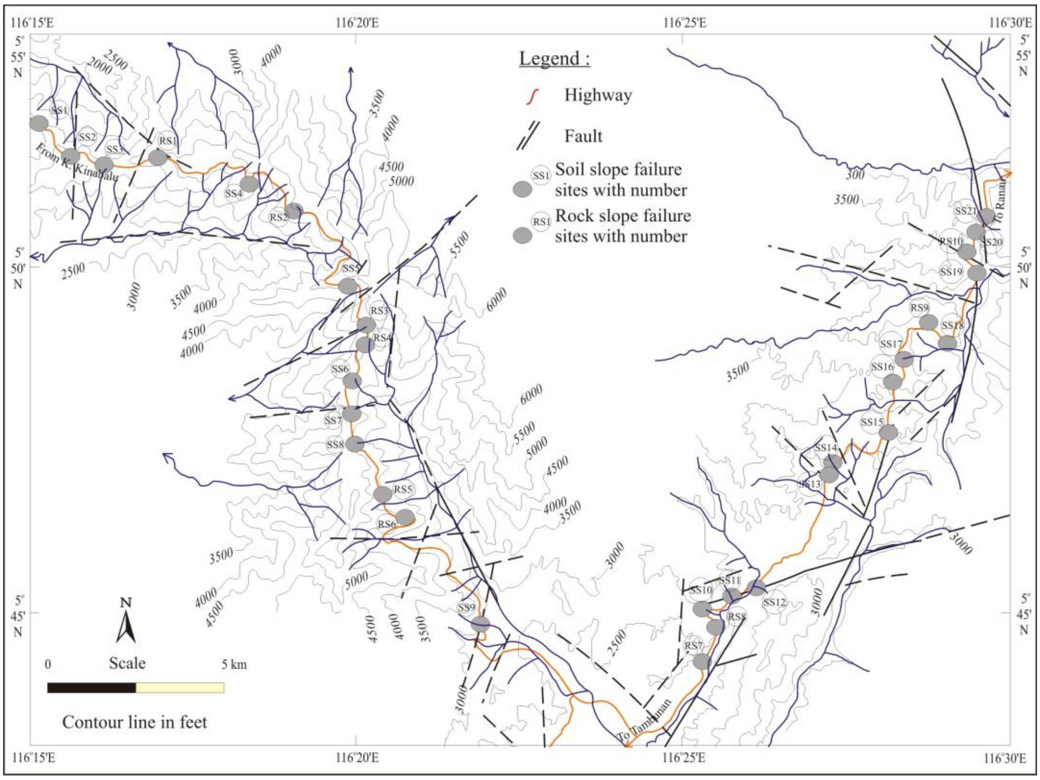



**Figure 2:** Location map of the slopes

## 3. LOCAL GEOLOGY AND ENGINEERING GEOLOGY CHARACTERISTICS

The geology of the study area is made up of sedimentary rock of the Crocker Formation (Late Eocene age) and Quaternary Alluvium Deposits. Table 1 shows the composite stratigraphic column of rock units with their water bearing and engineering properties. The effect of faulting activity can be observed on the lithologies of the study area. This was confirmed by the existence of transformed faulted material consisting of angular to sub angular sandstone fragments, with fine recrystallined quartz along the joint planes, poorly sorted sheared materials and marked by the occurrence of fault gouge with fragments of subphyllite and slickensided surfaces.

Engineering works may involves excavating, removing of the existing overburden soil and weathered rocks, filling of lowland and cutting of hill slope. These processes exposed the rock and soil in the study area to weathering and erosion. The slope materials become weak and loss its engineering properties. Moreover, the weathered products have high content of clay which may lower the rock strength and its engineering properties, approaching that of soil [10]. The cutting of hill slope, dumping of the stream within or along the slope and removing trees and vegetation from the hill slope reduces the stability of the slope. As a result, the hill slope becomes critical / unstable. The layered nature of the sandstone, siltstone and shale may constitute possible sliding surfaces (Table 1). The sandstone-shale contact is easily accessible by water and such contact seepage may weaken the shale surface and cause slides and falls within the formations. Interbedded sandstone, siltstone and shale may also present problems of settlement and rebound. The magnitude, however, depends on the character and extent of shearing in the shale. The strength of the sandstone will also depend on the amount and type of cement-matrix material occupying the voids. The sandstones are compacted and in grain to grain contact with each other. Instead of chemical cement (vein) or matrix, the pores are filled by finer-grained sands to silt-sized materials or squeezed rock fragments. The absence of chemical cement reduces the strength of the sandstone especially when it is weathered or structurally disturbed. The shale units have an adequate strength under dry conditions but lose this strength when wet [10]. During the rainy season, the shale becomes highly saturated with water which increases the water pressure and reduces resistances to sliding and falling especially within the sandstones-shale contact. This condition, in addition to varying amounts of bitumen and levels of degradation, makes shale unpredictable and unsuitable for road construction sites. Its unstable nature can be remedied by proper management of soaking and draining of water from the rock or along the sandstone-shale contact.

**Table 1:** Local Stratigraphic Column and their Water Bearing and Engineering Properties

| Age | Rock Formation | Unit | General Character | Water-Bearing Properties | Engineering Properties |
|---|---|---|---|---|---|
| Quaternary | Alluvium | - | Unconsolidated gravel, sand and silt with minor amounts of clay deposited along the rivers or streams and their tributaries. Includes natural levee and flood plain deposit. | Gravelly and sandy, portions are highly permeable and yield large quantities of water. Important to groundwater development. | Generally, poorly consolidated. Hence not suitable for heavy structures and subsidence under heavy load. |
| Late Eocene to Early Miocene | Crocker Formation | Shale | This unit is composed of two types of shale red and grey. It is a sequence of alteration of shale with siltstone of very fine. | It has no significant to groundwater development due to its impermeable characteristic. | Very dangerous site for heavy structures and the main causes of mass movement. |
| | | Interbedded Shale-Sandstone | It is a sequence of interlayering of permeable sandstone with impermeable shale. The permeability of this unit is quite variable. Groundwater in this unit tends to be under semi-confined to confined system. | Little importance to groundwater provides some water but not enough for groundwater development. | Dangerous site for heavy structures and high potential for mass movement. |
| | | Sandstone | Light grey to cream colour, medium to coarse -grained and some time pebbly. It is highly folded, faulted, jointed, fractured occasionally cavernous, surfically oxidized and exhibits spheriodal weathering. | Importance to groundwater. | Good site for heavy structures with careful investigation. Stable from mass movement and provide some modification like closing of continuous structure. |

## 4. GEOHYDROLOGIC AND HYDROGEOLOGY

The study area and its surrounding areas are controlled by heavy drainage system of different patterns (e.g. Trellis and Parallel) (Figure 2). The region has a high drainage density, being the cradle and origin of major rivers in the study area. Structurally, a number of linear river segments belong to different watershed systems indicate the existence of major tectonic fractures. The structural control of the river tributaries of the area is evidenced by the physical characteristics of sedimentary rocks; highly fractured areas and less competent shale beds. The sedimentary rocks are more intensely dissected by fault zones than the ultrabasic rocks.

Groundwater occurs and moves through interstices or secondary pore openings in the rock formations. Such openings can be the pore spaces between individual sedimentary grains, open joints and fractures in hard rocks or solution and cavernous opening in brecciated layers and cataclasites. The direction of groundwater movement is generally under the influence of gravity. The rock formations exhibit a high degree of weathering and covered by thick residual soil that extends to more than 30 meters in thickness. The weathered materials are weak and caused slope failures due to high fractured porosity and high pore pressure subjected by both shallow and deep groundwater.

Calculation of the groundwater balance helps to show the amount of rain water available for surface run-off and deep percolation. The Thornwaite's method is used to compute for the potential evapotranspiration (evaporation & transpiration). This method is selected because it needs only the temperature and rainfall data. Analysis from Table 2 shows that during the earlier (January to March) and middle (June to September) part of the year, the potential evapotranspiration is higher than rainfall. Theoretically, therefore all the precipitation that fall over the area during this period will be lost through evapotranspiration, leaving no excess water available for run-off and deep percolation. It is only in the months of April, May and later months of the year (October to December), that excess rainfall becomes available for run-off and deep percolation. Result show the high amount of evapotranspiration, reduced the water available for run-off and deep percolation to a more 40 mm per year. One would have to note in mind that the average surface temperature of the study area is lower than the temperature used in the computation of Table 2. Therefore, the true evapotranspiration could be significantly lower allowing more surface sun-off and deep percolation.

**Table 2:** Groundwater Balance for the year 2016

| **Month** | | **Jan.** | **Feb** | **March** | **April** | **May** | **June** | **July** | **August** | **Sept.** | **Oct.** | **Nov.** | **Dec.** | **Total** |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Temperature | | 26.80 | 27.04 | 27.51 | 28.11 | 28.11 | 27.95 | 29.81 | 27.51 | 27.34 | 27.33 | 27.08 | 26.86 | - |
| Monthly Temperature Index | | 12.68 | 12.89 | 13.23 | 13.64 | 13.67 | 13.56 | 13.43 | 13.24 | 13.12 | 13.00 | 12.93 | 12.75 | 158.14 |
| Standard Potential Evapotrans-piration (mm) | | 138.05 | 141.96 | 141.68 | 143.89 | 144.07 | 143.30 | 143.55 | 141.67 | 141.06 | 140.72 | 140.07 | 139.26 | 1700.28 |
| Correction Factor | | 1.02 | 0.93 | 1.04 | 1.02 | 1.06 | 1.04 | 1.06 | 1.06 | 1.02 | 1.03 | 1.00 | 1.02 | - |
| Evapotrans-piration Potential after correction | | 141.83 | 131.31 | 146.64 | 146.77 | 152.71 | 148.32 | 152.16 | 149.46 | 143.18 | 144.94 | 139.37 | 141.35 | 1738.04 |
| Rain (mm) | | 117.60 | 127.60 | 109.20 | 153.80 | 166.60 | 130.90 | 119.70 | 138.10 | 119.70 | 145.30 | 147.00 | 153.00 | 1628.50 |
| Difference | -ve | -24.23 | -3.71 | -37.44 | - | - | -17.42 | -32.46 | -11.36 | -23.48 | - | - | - | -150.10 |
| | +ve | - | - | - | 7.03 | 13.89 | - | - | - | - | 0.36 | 7.63 | 11.65 | - |
| True Evapotrans-piration before correction | | 117.60 | 127.60 | 109.20 | 146.77 | 152.71 | 130.90 | 119.70 | 138.10 | 119.70 | 144.94 | 139.37 | 141.35 | 1587.94 |

Yearly soil moisture deficiency = **150.1**
True yearly evapotranspiration = **1587.94**
Water available for seepage and run-off = **4.56 mm**

## 5. SLOPE STABILITY ANALYSIS

In this study, a total of 31 selected critical slope failures were studied and classified into two main groups: soil slope and rock slope. Failures in soil slopes (including embankments) are 21 (67 %) whereas 10 of all failures (33 %) of rock slope. Soil slope failures normally involved large volumes of failed material as compared much rock slopes, where the failures are mostly small. Of the 21 failures in soil slopes, 15 (71 %) are embankment failures making them 48 % of all types of failures.

Results of a detailed analysis of soil slope stability are presented in Table 3. Considering cut slopes, all the major lithologies are involved showing that this type of failure is not mostly controlled by lithology. The failure volume scale involved generally small to large in size possibly endangering road users. In term of weathering grades, the materials that underwent failure were in the ranges from grade IV to VI (Figures 3 to 6). Weathering is the main factor causing failure with the depth of weathering influencing the volume of material that fails. It appears that grade IV to grade V materials actually failed with the overlying grade VI material sliding or slumping down together with this material during failure. Physical and mechanical properties of 84 soil samples indicated that the failure materials mainly consist of poorly graded to well graded materials of clayey loamy soils, which characterized by low to intermediate plasticity content (9 % to 28 %), containing of inactive to normal clay (0.34 to 1.45), very high to medium degree of swelling (5.63 to 13.85), variable low to high water content (4 % to 22 %), specific gravity ranges from 2.57 to 2.80, low permeability (9.66 X 10-3 to 4.33 X 10-3 cm/s), friction angle (□) ranges from 7.70° to 29.20° and cohesion (C) ranges from 3.20 KPa to 17.27 KPa. Soil slopes stability analysis indicates that the factor of safety value as unsafe (0.56 to 0.98). The presence of ground water, slope angle, removal of vegetation cover, lack of proper drainage system, artificial changing, climatological setting, geological characteristics and material characteristics are additional factors contributing to the failures.

**Table 3:** Analysis results of soil slope failures

| *Type of failure* | **Shallow slide (T1 –a)** | | | **Deep slide (T1 – b)** | | **Multiple Slump (T2-b)** | | **Earth Flow (T3-a)** | |
|---|---|---|---|---|---|---|---|---|---|
| **Location (km)** | KM 12 | KM 35 | KM 61 | KM 17 | KM 45 | KM 26 | KM 82 | KM 15 | KM 88 |
| **Slope** | SS1 | SS5 | SS9 | SS3 | SS6 | SS4 | SS13 | SS2 | SS17 |
| *Geological Formation* | Crocker Formation | Crocker Formation | Crocker Formation | Crocker Formation | Crocker Formation | Crocker Formation | Crocker Formation | Crocker Formation | Crocker Formation |
| **Lithology** | Sediment | Sediment | Sediment | Sediment | Sediment | Sediment | Sediment | Sediment | Sediment |
| **Weathering grade** | IV to VI | IV to VI | IV to VI | IV to VI | IV to VI | IV to VI | IV to VI | IV to VI | IV to VI |
| **Volume (1)** | Small | Large | Medium | Large | Large | Small | Small | Small | Medium |
| **Sand (%)** | 76 – 78 | 60 – 63 | 65 – 68 | 45 – 48 | 68 – 70 | 21 – 24 | 44 – 45 | 55 – 58 | 62 – 63 |
| **Silt (%)** | 13 – 14 | 10 – 14 | 8 – 12 | 13 – 16 | 16 – 18 | 54 – 58 | 28 – 33 | 9 – 13 | 10 – 13 |
| **Clay (%)** | 22 – 24 | 26 – 28 | 22 – 24 | 38 – 40 | 15 – 16 | 20 – 22 | 20 – 23 | 31 – 33 | 22 – 26 |
| **Liquid limit (%)** | 37 – 39 | 27 – 29 | 27 – 30 | 31 – 33 | 28 – 32 | 31 – 34 | 27 – 31 | 25 – 29 | 33 – 38 |
| **Plastic limit (%)** | 21 – 23 | 15 – 16 | 12 – 14 | 16 – 19 | 17 – 18 | 12 – 14 | 14 – 17 | 15 – 18 | 16 – 19 |
| **Plasticity index (%)** | 15 – 18 | 12 – 15 | 13 – 16 | 15 – 17 | 15 – 17 | 19 – 22 | 17 – 19 | 14 – 16 | 17 – 20 |
| **Liquidity index (%)** | - 0.05 to - 0.03 | -0.39 to - 0.37 | 0.02 to 0.04 | - 0.11 to - 0.09 | - 1.56 to - 1.53 | - 0.35 to - 0.30 | 0.02 to 0.05 | - 0.18 to - 0.14 | 0.09 to 0.15 |
| **Clay activity** | 0.98 – 1.00 | 0.40 – 0.48 | 0.34 – 0.40 | 0.41 – 0.45 | 0.66 – 0.75 | 0.98 – 0.99 | 0.78 – 0.85 | 0.46 – 0.52 | 0.42 – 0.47 |

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| **Shrinkage limit (%)** | 8.45 – 8.50 | 5.79 – 5.88 | 6.10 – 6.35 | 7.28 – 7.55 | 7.04 – 7.65 | 8.68 – 8.89 | 7.98 – 8.25 | 6.34 – 677 | 7.98 – 8.33 |
| **Moisture content (%)** | 20 – 22 | 10 – 14 | 10 – 12 | 11 – 14 | 7 – 10 | 5 – 8 | 14 – 19 | 12 – 14 | 10 – 14 |
| **Specific gravity** | 2.65 – 2.68 | 2.63 – 2.64 | 2.76 – 2.78 | 2.60 – 2.62 | 2.73 – 2.77 | 2.60 – 2.62 | 2.66 – 2.67 | 2.68 – 2.72 | 2.74 – 2.80 |
| **Permeability (cm/s) (X $10^{-3}$)** | 8.54 | 7.55 | 9.15 | 6.39 | 8.28 | 3.32 | 9.08 | 7.98 | 7.40 |
| **Cohesion, C (kN/m$^2$)** | 7.20 | 7.31 | 6.78 | 9.50 | 6.27 | 5.13 | 12.29 | 12.54 | 3.20 |
| **Friction angle ($^o$)** | 26.30 | 29.20 | 28.90 | 25.50 | 22.90 | 7.70 | 17.30 | 9.30 | 21.00 |
| **Factor of Safety** | 0.87 | 0.97 | 0.95 | 0.78 | 0.76 | 0.65 | 0.68 | 0.89 | 0.91 |
| **Main factors causing failures** | SA, W, V, GWL, M, C, G, OBV, DS, EC and AC | | | | | | | | |

**Table 3:** *(Cont'd)* Analysis results of soil slope failures

| *Type of failure* | **Debris Flow (T3 – b)** | | **Creep (T4)** | | **Complex failure (Slide Flow) (T5 – a)** | | | |
|---|---|---|---|---|---|---|---|---|
| **Location (km)** | KM 48 | KM 75 | KM 83 | KM 92 | KM 50 | KM 76 | KM 85 | KM 100 |
| **Slope** | SS7 | SS10 | SS14 | SS18 | SS8 | SS11 | SS15 | SS20 |
| *Geological Formation* | Crocker Formation | Crocker Formation | Crocker Formation | Crocker Formation | Crocker Formation | Crocker Formation | Crocker Formation | Crocker Formation |
| **Lithology** | Sediment | Sediment | Sediment | Sediment | Sediment | Sediment | Sediment | Sediment |
| **Weathering grade** | IV to VI | IV to VI | IV to VI | IV to VI | IV to VI | IV to VI | IV to VI | IV to VI |
| **Volume (1)** | Medium | Medium | Large | Medium | Medium | Medium | Medium | Medium |
| **Sand (%)** | 54 – 58 | 20 – 23 | 44 – 45 | 68 – 70 | 59 – 61 | 63 – 65 | 36 – 39 | 39 – 42 |
| **Silt (%)** | 21 – 23 | 52 – 55 | 16 – 19 | 12 – 16 | 6 – 10 | 5 – 12 | 22 – 26 | 18 – 20 |
| **Clay (%)** | 20 – 22 | 20 – 22 | 32 – 35 | 18 – 22 | 30 – 33 | 32 – 36 | 38 – 40 | 40 – 43 |
| **Liquid limit (%)** | 31 – 33 | 28 – 32 | 39 – 41 | 28 – 30 | 26 – 30 | 28 – 31 | 41 – 44 | 41 – 43 |
| **Plastic limit (%)** | 13 – 16 | 13 – 15 | 16 – 19 | 16 – 20 | 10 – 14 | 15 – 18 | 22 – 24 | 23 – 25 |
| **Plasticity index (%)** | 18 – 20 | 15 – 19 | 20 – 23 | 9 – 12 | 12 – 18 | 12 – 16 | 17 – 19 | 18 – 20 |
| **Liquidity index (%)** | - 0.02 to - 0.01 | - 0.33 to - 0.25 | 0.14 to 0.18 | - 0.62 to - 0.58 | - 0.88 to - 0.85 | - 0.68 to – 0.60 | - 0.84 to - 0.78 | - 0.85 to - 0.83 |
| **Clay activity** | 0.87 – 0.91 | 1.00 – 1.11 | 0.53 – 0.55 | 0.48 – 0.50 | 0.38 – 0.39 | 0.47 – 0.50 | 0.43 – 0.49 | 1.43 – 1.45 |
| **Shrinkage limit (%)** | 8.53 – 9.12 | 9.16 – 9.86 | 8.84 – 9.98 | 5.63 – 6.53 | 5.63 – 6.66 | 7.51 – 7.95 | 7.98 – 8.65 | 8.45 – 9.26 |
| **Moisture content (%)** | 13 – 15 | 6 – 10 | 22 – 25 | 7 – 12 | 4 – 8 | 4 – 8 | 9 – 12 | 7 – 11 |
| **Specific gravity** | 2.61 – 2.63 | 2.61 – 2.64 | 2.60 – 2.62 | 2.72 – 2.77 | 2.66 – 2.68 | 2.57 – 2.58 | 2.64 – 2.68 | 2.62 – 2.69 |
| **Permeability (cm/s) (X $10^{-3}$)** | 5.41 | 5.60 | 5.66 | 9.66 | 8.78 | 7.83 | 4.33 | 5.58 |
| **Cohesion, C (kN/m$^2$)** | 11.43 | 17.27 | 7.76 | 9.62 | 10.40 | 9.82 | 12.80 | 15.47 |
| **Friction angle ($^o$)** | 11.29 | 23.70 | 27.70 | 21.20 | 24.50 | 29.50 | 21.50 | 22.30 |
| **Factor of Safety** | 0.85 | 0.56 | 0.88 | 0.89 | 0.98 | 0.78 | 0.58 | 0.63 |
| **Main factors causing failures** | SA, W, V, GWL, M, C, G, OBV, DS, EC and AC | | | | | | | |

**Table 3:** *(Cont'd)* Analysis results of soil slope failures

| **Type of failure** | **Complex failure (Slump flow) (T5 – b)** | | | |
|---|---|---|---|---|
| **Location (km)** | KM 77 | KM 87 | KM 96 | KM 101 |
| **Slope** | SS12 | SS16 | SS19 | SS21 |
| *Geological Formation* | Crocker Formation | Crocker Formation | Crocker Formation | Crocker Formation |
| **Lithology** | Sediment | Sediment | Sediment | Sediment |
| **Weathering grade** | IV to VI | IV to VI | IV to VI | IV to VI |
| **Volume (1)** | Large | Large | Large | Large |
| **Sand (%)** | 48 – 51 | 46 – 50 | 46 – 47 | 48 – 51 |
| **Silt (%)** | 18 – 22 | 18 – 20 | 10 – 13 | 7 – 11 |
| **Clay (%)** | 26 – 30 | 30 – 33 | 36 – 38 | 38 – 42 |
| **Liquid limit (%)** | 31 – 33 | 35 – 37 | 46 – 49 | 29 – 32 |
| **Plastic limit (%)** | 14 – 16 | 22 – 25 | 19 – 21 | 13 – 17 |
| **Plasticity index (%)** | 17 – 19 | 13 – 15 | 27 – 28 | 18 – 21 |
| **Liquidity index (%)** | - 0.26 to - 0.22 | - 1.08 to - 1.05 | - 0.41 to - 0.38 | 0.18 to 0.20 |
| **Clay activity** | 0.62 – 0.68 | 0.35 – 0.40 | 0.69 – 0.77 | 0.38 – 0.47 |
| **Shrinkage limit (%)** | 7.98 – 8.12 | 6.10 – 7.33 | 12.68 – 13.85 | 8.22 – 10.54 |
| **Moisture content (%)** | 9 – 12 | 8 – 10 | 6 – 10 | 14 – 17 |
| **Specific gravity** | 2.60 – 2.62 | 2.64 – 2.65 | 2.58 – 2.60 | 2.65 – 2.68 |

| | | | | |
|---|---|---|---|---|
| **Permeability (cm/s) (X 10$^{-3}$)** | 7.81 | 7.61 | 4.62 | 8.47 |
| **Cohesion, C (kN/m$^2$)** | 10.40 | 10.36 | 11.43 | 8.53 |
| **Friction angle (°)** | 24.50 | 18.50 | 11.29 | 20.45 |
| **Factor of Safety** | 0.87 | 0.95 | 0.79 | 0.92 |
| **Main factors causing failures** | SA, W, GWL, M, C, G, DS and AC | | | |

**Note:** (1) Volume: small (10 – 50 m$^3$), Medium (50 – 500 m$^3$) and Large (> 500 m$^3$) and (2) Discontinuity (D), Slope angle (SA), Weathering (W), Vegetation (V), Groundwater level (GWL), Material characteristics (M), Climatological setting (C), Geological characteristics (G), Over burden or vibration (OBV), Drainage system (DS), Embankment construction (EC) and Artificial changing (AC)

**Figure 3:** Shallow slide (T1 – a) at KM 35 (SS5) shows the failure movement are starting to move into several discrete blocks through the development of transverse cracks

**Figure 4:** Embankment failure in the form of a deep slide (T1 – b) at KM 45 (SS6)

**Figure 5:** Earth flow (T3-a) showing the settling soil block suffers from fracturing, stumping or flowing considerable lateral movement along the basal mobile zone is common, as is upheaval of the terrain down slope of the failure (Location: KM 88 (SS17)

**Figure 6:** Complex failure (slump flow) (T5 – b) at KM96 (SS19) shows the failure movement are starting to move into several discrete blocks through the development of transverse cracks

Table 3 shows the results of a detailed analysis of rock slope failures. Although rock slope failures contributed only 33 % (10 failures) of the total failures, they involved large volume of weathered and brecciated rocks (Figures 7 & 8). The main factor contributing to rock slope failures was the orientation and intensity of discontinuity planes. That is why rock slope failures occur most frequently along the highway on sedimentary rocks, which were highly brecciated and fractured. Generally, the failed material underwent only moderately to completely weathering (grade III to V). The rock properties characterization for 10 rock samples indicated that point load strength index ranges from 0.33 MPa to 0.52 MPa (moderately week) and uniaxial compressive strength range from 7.81 MPa to 12.57 MPa (moderately week). Kinematics slope analyses indicates that the variable potential of circular, planar, wedges and toppling failures modes as well as the combination of more than one mode of aforementioned failure (Figure 9). Rock slopes stability analysis indicates that the factor of safety value as unsafe (0.52 to 0.95). Other factors contributing to rock slope failure are the presence of groundwater, climatological setting, joints filling material, high degree of rock fracturing due to shearing, steep of slope angle, high intensive of faulting and folding activities and locating at the fault zones area.

**Table 4:** Analysis results of rock slope failures

| Location (km) | Slope | Geological formations | Lithology | Weathering grade | Slope face orientation (°) | Volume (1) | Major Discontinuities (°) | Intersect involved | Critical | Release | Potential | Possible | Point load strength index, IS (50) (MPa) | Uniaxial compressive strength correlation, UCS = 24 IS (50) (MPa) | Factor of Safety | Main factors causing failures (2) | Mitigation measure |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| KM 21 | RS1 | Crocker Formation | Sediment | III to V | 110/74 | Large | J1=049/44, J2=164/42, J3=250/34, J4=305/27, J5=343/51, J6=111/30 & J7=210/36 | J6 | J6 | J1, J2, J5 & J7 | Plane | Circular | 0.33 | 7.81 | 0.88 | D, SA, W, GWL, MC, WR, CR, BF, D, GC & AC | RB, RD, S & RHD |
| | | | | | | | | J4 | J4 | - | Toppling | - | | | | | |
| KM 29 | RS2 | Crocker Formation | Sediment | III to IV | 190/60 | Large | J1=055/53, J2=172/47, J3=090/68, J4=211/62 & J5=309/27 | J3 X J4<br>J2 X J3 | J3 | J1, J2, J4 & J5 | Wedge | - | 0.42 | 10.11 | 0.74 | | |
| | | | | | | | | J2 X J4 | J2 | J3 & J4 | | | | | | | |
| KM 38 | RS3 | Crocker Formation | Sediment | III to IV | 050/65 | Medium | J1=016/29, J2=071/39, J3=301/67, J4=168/67, J5=240/70, J6=138/75 & J7=115/37 | J2 X J6<br>J2 X J4 X J7 | J6<br>J4 | J3 & J5<br>J2 & J6 | Wedge | Circular | 0.42 | 9.88 | 0.92 | | |
| | | | | | | | | J5 | J5 | J1, J3, J4, J6 & J7 | - | Toppling | | | | | |
| KM 41 | RS4 | Crocker Formation | Sediment | III to IV | 308/75 | Medium | J1=154/45, J2=062/47 & J3=303/35 | J3 | J3 | J1 & J2 | Plane | - | 0.41 | 9.74 | 0.65 | | |
| KM 54 | RS5 | Crocker Formation | Sediment | III to IV | 275/70 | Large | J1=096/17, J2=161/17 & J3=270/63 | J3 | J3 | J1 & J2 | Plane | - | 0.47 | 11.37 | 0.77 | | |
| KM 56 | RS6 | Crocker Formation | Sediment | III to IV | 125/78 | Medium | J1=063/53, J2=174/46, J3=041/74, J4=214/61, J5=306/29, J6=266/54 & J7=163/68 | J3 X J7<br>J1 X J3<br>J1 X J2<br>J2 X J3 | J3<br>J1<br>J2<br>J2 | J4 & J5<br>J1, J3 & J5<br>J1, J4 & J7<br>J1, J3 & J7 | Wedge | Circular | 0.48 | 11.64 | 0.89 | | |
| | | | | | | | | J6 | J6 | J1, J2, J3 & J7 | - | Toppling | | | | | |
| KM 72 | RS7 | Crocker Formation | Sediment | III to IV | 150/60 | Large | J1=023/25, J2=148/47, J3=300/65 & J4=224/48 | J2 X J4 | J4 | J1, J2 & J3 | Wedge | - | 0.47 | 11.27 | 0.52 | | |
| | | | | | | | | J2 | J2 | J1, J3 & J4 | Plane | - | | | | | |
| | | | | | | | | J3 | J3 | J1 & J4 | - | Toppling | | | | | |
| KM 74 | RS8 | Crocker Formation | Sediment | III to IV | 010/68 | Large | J1=145/34, J2=306/66, J3=014/76 & J4=064/71 | J2 X J4 | J2 | J3 & J4 | Wedge | - | 0.52 | 12.57 | 0.95 | | |
| | | | | | | | | J3 | J3 | J1, J2 & J4 | - | Plane | | | | | |
| KM 90 | RS9 | Crocker Formation | Sediment | III to V | 025/78 | Large | J1=171/45, J2=066/68, J3=301/66, J4=018/76, J5=241/75, J6=339/75 & J7=307/30 | J2 X J3<br>J2 X J6 | J3<br>J2 | J2 & J6<br>J4 & J6 | Wedge | Circular | 0.47 | 11.34 | 0.68 | | |
| | | | | | | | | J4 | J4 | J1, J2 & J5<br>J3, J4 & J6 | Plane | - | | | | | |
| KM 98 | RS10 | Crocker Formation | Sediment | III to IV | 055/70 | Medium | J1=052/42, J2=141/41, J3=302/71 & J4=247/73 | J1 X J3<br>J1 X J2 | J3<br>J2 | J1, J2 & J4<br>J1, J3 & J4 | Wedge | - | 0.44 | 9.95 | 0.76 | | |
| | | | | | | | | J1 | J1 | J2, J3 & J4 | Plane | - | | | | | |

- **Note *(Cont'd)***

(1) Volume: small (10 – 50 $m^3$), Medium (50 – 500 $m^3$) and Large (> 500 $m^3$)
(2) Discontinuity (D), Slope angle (SA), Weathering (W), Groundwater level (GWL), Material characteristics (MC), Weak rock (WR), Crushed rock (CR), Blocks and fragments (BF), Debris (D), Geological characteristics (GC) and artificial changing (AC)
(3) Rock bolt (RB), Rock dowel (RD), Shotcrete (S) & Rock horizontal drainage (RHD)

**Figure 7:** Circular failure (crushed) (B1 – b) at East KM 21 (RS1) shows the surface weakness (revealing) within the structure, which composed of coherent crushed rocks on the gently sloping discontinuities

**Figure 8:** Wedge failure (blocks and fragments) (B3 – a) at KM 29 (RS2) indicates the form of discontinuities provides intersecting sheets to form yield stepped surfaces or boundary vertical joints

## 6. DISCUSSION

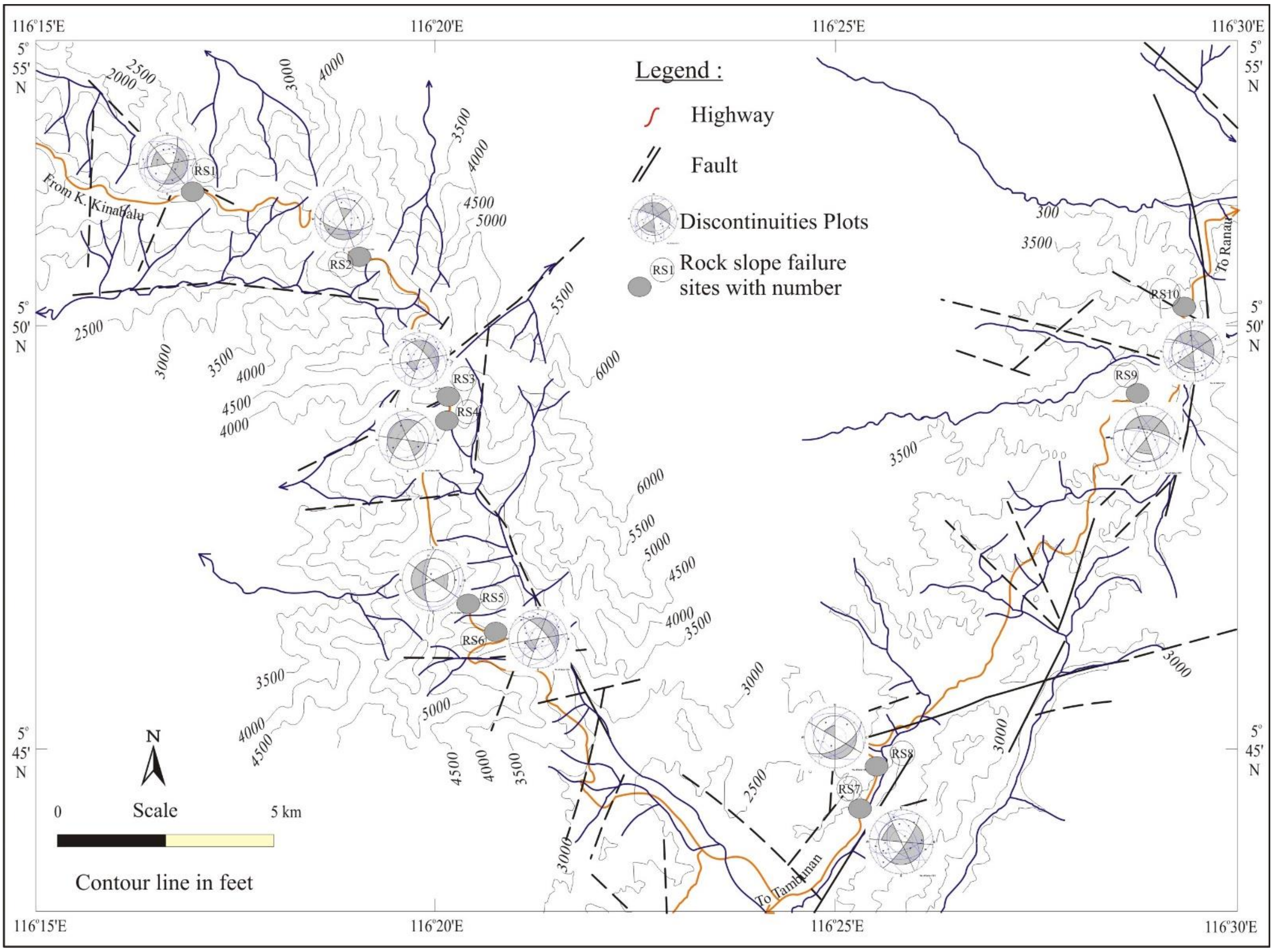



**Figure 9:** Streoplots and view of rock slope failures in the study area

The steep topography terrain in the study area is naturally slope instability prone areas. Drainage systems in this steep rugged terrain are characterised by short and rapid flowing streams. These fast-moving bodies of water, causes surface erosion and gulling on the slopes. Surface erosion removes the necessary top soil to sustain vegetation cover. This further exposes the slope and weakens the strength of the slope materials. The steep terrain also poses real problems for infrastructural developments, like road construction. Due to this reason, construction of roads across the mountain range would definitely involve slope cutting and building of fill slopes.

The geology of the Crocker Formation does not benefit slope stability. The interbedded sandstone-shale and shale unit lithologies of the Crocker Formation weather, rapidly when exposed to the elements. This is especially through along many man-made slope cuts. Weathering changes the sandstone and shale into fine clayey materials. Determined indirectly from their clay activity values some of these clay minerals are suspected to be in-active and normal clay [11]. These two types of clay minerals when interacting with water would expand and lubricate rock joints and other discontinuities. The highly fractured nature of the lithology also contributes to slope instability. Fractured rock masses have much lower shear strength compared to the original fresh rock. The orientation of the discontinuities and its relation to the geometry and strike of the slope, have a direct influence on the occurrence of rock slope failures. Due to the influence of the regional tectonic forces the rock joints in the study area are predominantly orientated at a Northeast - Southwest and Northwest – Southeast direction. It has been determined that slopes that strike northward, westward, and South – Southeast show a higher probability to fails. Therefore, as a matter of precaution, cutting the slopes at these strike directions, if possible should be avoided.

The climate of the study area is very much the same as the other parts of Sabah, Malaysia. The condition of a warm and moist tropical climate induces rapid weathering of the lithology forming thick weathering failure of fill slopes. Due to the rugged topography sections of the road profiles. High annual rainfall is accumulated during periods of rainfall and drizzling which occur daily in this high mountainous region. The high amount of rainfall helps to sustain the moist and wet nature of the slope throughout most of the day. During the storm's period of high rainfall intensity, the rate of water infiltration to the soil would not be sufficient, the excess water would accumulate to form surface run off. Calculation on the groundwater balance shows that the area experiences high evapotranspiration, leaving only a small amount of water available for run off during most periods of the year [12]. During intense storms, the amount of run off and deep percolation greatly exceeds the evapotranspiration. Excessive surface run off causes rapid surface erosion, which blocks the limited existing man-made drainage. As the result of blocked and insufficient drainage the excess water overflows the drains and on to the road surface. The excess water would also accumulate on certain parts of the road and fill slope. At several locations where the huge drainage pipes are buried underneath the road, subsidence is seen to occur. Where the outlet of the drainage pipe (just below road level) directs the water down slope, erosion and undercutting of the fill slope is seen to occur. This shows a weakness in the drainage design. The drainage pipes should be extended to the nearest natural drainage.

Removal of vegetation cover on the slope will seriously reduce the slope stability. The natural vegetation on the slope cuts have almost all been removed. It has been with limited success replaced by secondary trees and grasses. The lost of the original anchoring buttress roots is irreplaceable. The younger plants have found it hard to grow on the thin layer of soil on the surface of slope cuts. Removal of vegetation also disrupts the hydrological cycle, by allowing more infiltration and reducing the removal of groundwater by transpiration. More infiltration and less transpiration would cause the soil materials to be more saturated with water. This weakens the strength of the slope directly. Another important contribution to slope failure is the anthropological factor. Man is solely responsible in the faulty design and construction of unstable slope cuts. In the study area man's road construction is responsible for much of the were built across natural drainage valleys. Without proper drainage, the fill slope blocks the path of water flow. The retention of water at these slopes, caused subsidence and tension cracks to appear on the road surface [11]. If left unchecked, total failure would occur at these sections of road. Lack of maintenance is another real problem; which slopes are left without proper care until landslides have occurred. Other contributing anthropological factor includes the heavy traffic flow, illegal deforestation of slopes and irresponsible slope land development.

Factor of safety (FOS) provides away for an engineering assessment of slope stability. Slope stability analyses were conducted for various conditions such as the existence of tension cracks and variation of shear strength parameters in order to study the dominant factors causing the slope failure. Rock and soil slopes stability analysis indicates that the FOS value as unsafe (0.52 to 0.98). The location of slip circles also tallies well with the slope failures as observed in the field. This confirms that the mobilised strength during the slope failure is very close to the subsoil peak strength parameter interpreted from the laboratory strength test results. Any rise in groundwater profile would certainly further reduce the FOS. In dealing with risk of slope failure, level of awareness and mitigation measures must be increased when there is obvious climatic change especially on rainy season.

## 7. CONCLUSIONS

In light of available information, the following conclusions may be drawn from the present study:

1. A total of 31 selected critical slope failures were studied. Failures in soil slopes (including embankments) are 21 (67 %) whereas 10 of all failures (33 %) of rock slope. Soil slope failures normally involved large volumes of failed material as compared much rock slopes, where the failures are mostly small. Of the 21 failures in soil slopes, 15 (71 %) are embankment failures making them 48 % of all types of failures.

2. Physical and mechanical properties of 84 soil samples indicated that the failure materials mainly consist of poorly graded to well graded materials of clayey loamy soils, which characterized by low to intermediate plasticity content (9 % to 28 %), containing of inactive to normal clay (0.34 to 1.45), very high to medium degree of swelling (5.63 to 13.85), variable low to high water content (4 % to 22 %), specific gravity ranges from 2.57 to 2.80, low permeability ($9.66 \times 10^{-3}$ to $4.33 \times 10^{-3}$ cm/s), friction angle (ϕ) ranges from 7.70˚ to 29.20˚ and cohesion (C) ranges from 3.20 KPa to 17.27 KPa.

3. The rock properties of 10 rock samples indicated that the point load strength index and the uniaxial compressive strength range classified as moderately week. Kinematics slope analyses indicates that the variable potential of circular, planar, wedges and toppling failures modes as well as the combination of more than one mode of aforementioned failure.

4. Rock and soil slopes stability analysis indicates that the factor of safety value as unsafe (0.52 to 0.98).

5. Engineering geologic evaluation of the study area indicates that the slope failures took place when rock and soil materials were no longer able to resist the attraction of gravity due to a decrease in shear strength and increase in the shear stresses due to internal and external factors. Internal factors involve some factors change in either physical or chemical properties of the rock or soil such as topographic setting, climate, geologic setting and processes, groundwater condition and engineering characteristics. External factors involve increase of shear stress on slope, which usually involves a form of disturbance that is induced by man includes removal of vegetation cover, induced by vehicles loading and artificial changes.

## REFERENCES


[1] Komoo, I. 1985. Pengelasan Kegagalan Cerun di Malaysia. Jurnal Ilmu Alam, 14-15.

[2] British Standard BS 5930. 1981. Site Investigation. London: British Standard Institution.

[3] British Standard BS 1377. 1990. Methods of Test for Soils for Civil Engineering Purposes. London: British Standard Institution.

[4] ISRM. 1979a. Suggested methods for determining water content, porosity, density, absorption and related properties, and swelling and slake-durability index properties. ISRM Commission on Standardization of Laboratory and Field Tests. International Journal of Rock Mechanics and Mining Sciences, 16, 141-156.

[5] ISRM. 1979b. Suggested methods for determining the uniaxial compressive strength and deformability of rock materials. ISRM Commission on Standardization of Laboratory and Field Tests. International Journal of Rock Mechanics and Mining Sciences, 16, 135-140.

[6] ISRM. 1985. Suggested methods for determining point load strength. ISRM Commission on Standardization of Laboratory and Field Tests. International Journal of Rock Mechanics and Mining Sciences, 22 (2), 51-60.

[7] Puspito, N.T. 2004. Geophysical Hazards in Indonesia. International Seminar on the Active Geosphere. Institut Teknologi Bandung. Indonesia.

[8] Geoslope International Ltd. 2002. Slope/W User's guide for slope stability analysis. Version 5., Calgary, Alta, Canada.

[9] Watts, C.F. 2003. Rockpack III for Windows. ROCK Slope Stability Analysis Package. User's Manual. Radford Computerized University, Radford, Virginia

[10] Roslee, R., Tahir, S., Omang, S.A.K.S. 2006. Engineering Geology of the Kota Kinabalu Area, Sabah, Malaysia. Bulletin of the Geological Society of Malaysia, 52, 17-25.

[11] Edward Voo, L.Z. 1999. Slope Stability Analysis Along Kota Kinabalu – Tambunan Road, Sabah. M. Sc. Thesis, University Malaysia Sabah (Unpublished).

[12] Faisal, M.M., Tahir, S., Zan, E.V.L. 1998. Preliminary report on slope stability of the Kota Kinabalu Tambunan Road, Sabah. Borneo Science, 4, 11-26.