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APPLICATION OF GSI SYSTEM FOR SLOPE STABILITY STUDIES ON SELECTED SLOPES OF THE CROCKER FORMATION IN KOTA KINABALU AREA, SABAH

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ABSTRACT

This study was conducted on two selected slopes in Kota Kinabalu area of Sabah. The area is underlain by Crocker Formation which consisting of interbedded sandstone and shale layers. The objectives of this study are to determine the Geological Strength Index (GSI) rating, rock mass properties and slope stability for the selected slopes. Engineering geological mapping and discontinuity survey were conducted to obtain quantitative description of discontinuities as well as rock sampling based on grain sizes. GSI rating and disturbance factor was obtained from discontinuity survey and field observation on the slope face, respectively. Residual GSI rating was determined using empirical method. Laboratory study was done to determine the Uniaxial Compressive Strength via point load test and unit weight by dry density test along with the intact rock constant. Rock mass properties such as cohesion, friction angle, tensile strength, Young's modulus and residual strength were determined by applying GSI system into the Hoek-Brown criterion. Kinematic analysis and finite element analysis were conducted to identify localised mode of failure and the safety factor of the selected slopes. Prescriptive measures were used to determine the rock cut slope designs. GSI rating for both slopes were obtained with both slopes can be considered as stable according to kinematic analysis and finite element analysis. Prescriptive measures for slope protection are needed to prevent water pressure build up and future failure.

1. INTRODUCTION

This study was conducted on two selected slopes, namely slope A and slope B in Kota Kinabalu area, Sabah (Figure 1). The study area is underlain by Crocker Formation of Late Eocene to Late Early Miocene ages (Sanudin & Baba, 2007) which consisting of interbedded sandstone and shale layers. The objectives of this study are to determine the Geological Strength Index (GSI) rating, rock mass properties and slope stability for the selected slopes.

Geological Strength Index (GSI) was introduced by Hoek et al. (1992) as an extension from Hoek-Brown criterion. Both GSI and Hoek-Brown criterion were further refined by Hoek (1994), Hoek et al. (1995), and Hoek and Brown (1997). Marinos and Hoek (2002) further extend the application of GSI to heterogeneous rock mass such as flysch while dealing with incredibly difficult materials encountered in tunnelling in Greece and was further revised by Marinos (2007). The main purpose of GSI was to remove the dependent on Rock Quality Designation (RQD) (Deere et al., 1967) since RQD in most weak rock is essentially zero or meaningless, it became necessary to consider an alternatives classification system such as GSI. Since GSI classification is an extension from Hoek-Brown criterion, its applicability only limited to "isotropic" rock masses. Hoek et al. (2013) classified rock slope mass into three groups namely Group I, Group II and Group III as shown in Figure 2 which shows the transition from an isotropic intact rock (Group I), through a highly anisotropic rock mass (Group II) and to a heavily jointed rock mass (Group III) which can be considered as isotropic.

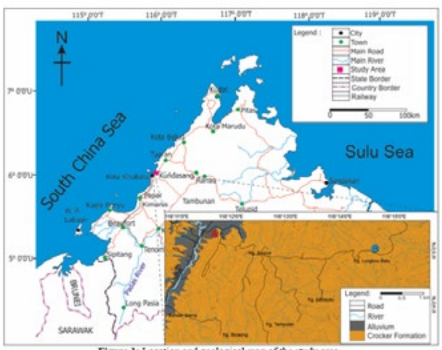

Figure 1: Location and geological map of the study area.

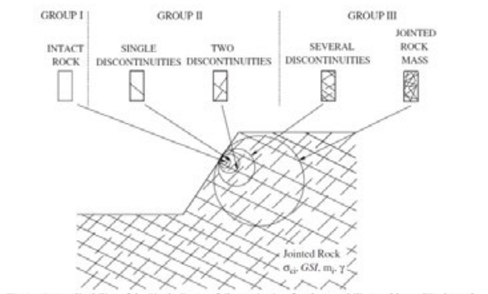

Figure 2: Applicability of the Hoek–Brown failure criterion for slope stability problems (Hoek *et al.*, 2013).

Finite element analysis (FEA) has been around for some time and was a numerical method for predicting real life effect through solving given equation that was subdivided from a domain into simpler part and a detailed review of slope stability analysis was presented by Duncan (1996) and Duncan's review of finite element analysis of slope concentrated mainly on deformation rather than stability analysis. Giffiths and Lane (1999) then discussed the elasto-plastic analysis using finite element method for slope stability analysis. Shear strength reduction (SSR) technique was used to determine the factor of safety of a slope. Hammah et al. (2004) examined the application of finite element analysis to determine the factor of safety of rock slope which strength was modelled by Generalized Hoek-Brown failure criterion and Hammah et al. (2005) introduced the development of SSR framework for the Hoek-Brown criterion.

Methodology

Engineering geological mapping and discontinuity survey were conducted to obtain quantitative description of discontinuities (ISRM, 1978) as well as rock sampling based on grain sizes. GSI rating (Marinos, 2007) and disturbance factor was obtained from discontinuity survey and field observation on the slope face, respectively. Surface condition (Bieniawski, 1989) along with the type of structure and composition of studied slope were identified and interpreted to determine the GSI rating. Residual GSI rating was determined using empirical method by Cai et al. (2007) which enable the determination of residual strength of the rock mass through Hoek-Brown criterion.

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Laboratory study was done to determine the Uniaxial Compressive Strength (UCS) via point load test (ISRM, 1985) and unit weight by dry density test (ISRM, 1979). The final UCS and dry density values of rock mass were obtained based on lithological unit thickness approach (Ismail Abd Rahim et al., 2009). Intact rock parameter (mi) for siltstone and shale unit was based on the suggested values given by Marinos and Hoek (2000). For sandstone, mi was obtained via empirical method by Shen and Karakus (2014). Rock mass properties such as cohesion, friction angle, tensile strength, Young's modulus and residual strength were determined by applying GSI system into the Hoek-Brown criterion which was computed using RocLab software (Rocscience, 2013). Kinematic analysis was done via Dips software (Rocscience Inc., 2004) to identify localised mode of failure. FEA was conducted to identify localised mode of failure and the safety factor of the selected slopes via Phase2 software (Rocscience Inc., 2013). Prescriptive measures (Yu et al., 2005) were used to determine the rock cut slope designs based on the result of both kinematic analysis and FEA.

Results and discussion

The GSI rating obtained for slope A is 38 which consists of interbedded of thick shale and sandstone layers (Photograph 3). It has 27m height, 24.03m length, 5.5m bench height, 1.2m bench width, 330°N slope face orientation, and 50° slope angle. Slope B has GSI rating of 43 and consists of interbedded of siltstone and shale with similar amount (Photograph 4). It has 21m height, 21.13m length, 5.2m bench height, 1.2m bench width, 270°N slope face orientation, and 50° slope angle. Figure 3 shows the GSI rating for slope A and slope B based on the chart for heterogeneous rock masses.

Photograph 3: Rock cut slope for slope A. Interlayer of thick shale and sandstone layers (Photograph orientation N 228°).

Photograph 4: Rock cut slope for slope B. Interlayer of sandstone and shale with similar amount (Photograph orientation N 182°).

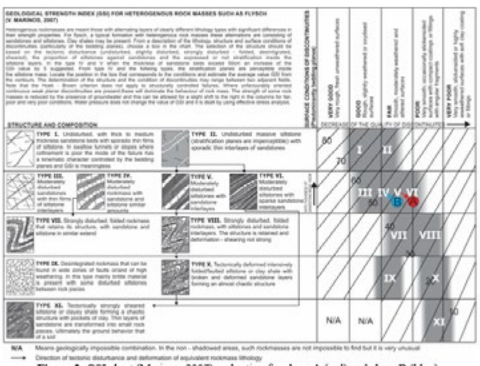

Figure 3: GSI chart (Marinos, 2007) and rating for slope A (red) and slope B (blue).

Table 1 shows the result of the parameters and rock mass properties for the selected slopes. Result shows that the rock mass properties of the rock masses were not only influenced by GSI rating but also mi and UCS as well. Even slope B which has higher GSI rating by comparing to slope A, slope B still has the weaker rock mass strength due to its weaker siltstone unit. The residual strength for the selected slopes was shown in Table 2. Following the trend of results in Table 1, slope B has the weaker rock mass properties in comparison with slope A.

Table 1: Parameter and rock mass properties for slope A and slope B.

Slope	Type	GSI	D	UCS (MPa)	Dry density (g/cm³)	m_i	Cohesion (MPa)	Friction angle (°)	Tensile strength (MPa)	Young's modulus (MPa)
A	V	38	0.7	46.31	2.48	7.82	0.166	39.22	0.022	2216.9
B	IV	43	0.7	29.71	2.45	6.19	0.135	38.25	0.028	2367.9

Table 2: Residual strength for slope A and slope B.

Slope	Residual GSI	Residual cohesion (MPa)	Residual friction angle (°)	Residual tensile strength (MPa)	Residual Young's modulus (MPa)
A	23	0.093	30.45	0.006	934.9
B	24	0.064	27.89	0.005	793.2

Kinematic analysis shows slope A (Figure 4 and Table 3) and slope B (Figure 5 and Table 4) do not have any potential mode of failure. This result was proven by the lack of structurally controlled failure on site. The number of discontinuity plane sets of slope A and slope B as shown in the stereonet enable the slopes to be considered as "isotropic" and its slope failure will not structurally controlled by discontinuity plane. Safety factor obtained from FEA for the slope A and slope B were 1.84 (Figure 6) and 1.74 (Figure 7), respectively. Both selected slopes can be considered as stable at the present time and it should be noted that both slope was assume dry during computation. Even though both slopes are stable now, the installation of wire mesh, bolting, weep holes, and surface drainage are needed to prevent future failure. The main purpose of these prescriptive measures for slope protection was to prevent water pressure build up within the slope since water is the main culprit triggering slope failure for tropical country such as Malaysia.

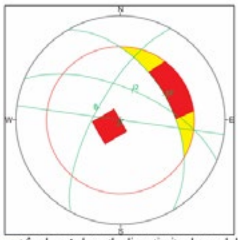

Figure 4: Stereonet for slope A shows the discontinuity plane and slope face plane.

Table 3: Potential mode of failure for slope A.

Slope face strike and dip		330/50	
Optimum slope angle		-	
Critical plane		-	
Discontinuity plane set	Strike and dip	Mode of failure	Possibility
B	209/70	Toppling	No
J1	98/89	Toppling	No
J2	293/65	Planar	No

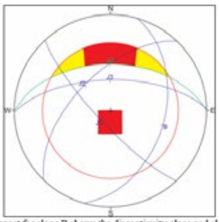

Figure 5: Stereonet for slope B shows the discontinuity plane and slope face plane.

Table 4: Potential mode of failure for slope B.

Slope face strike and dip		270/50	
Optimum slope angle		-	
Critical plane		-	
Discontinuity plane set	Strike and dip	Mode of failure	Possibility
B	16/43	Toppling	No
J1	137/79	Toppling	No
J2	224/59	Planar	No
J3	269/64	Planar	No

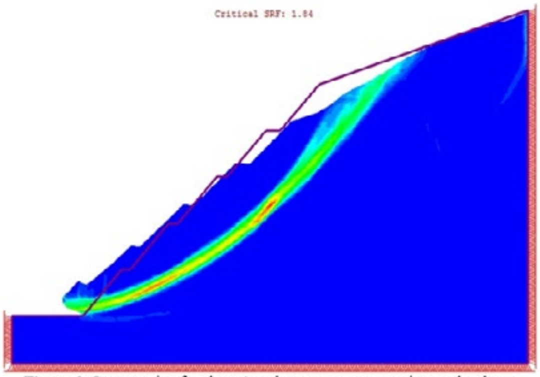

Figure 6: Cross section for slope A and mass movement acting on the slope.

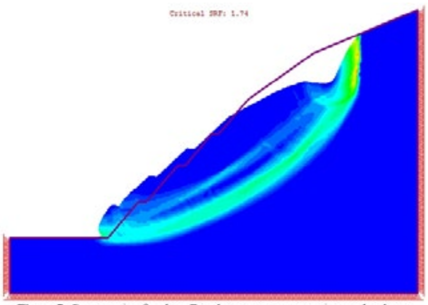

Figure 7: Cross section for slope B and mass movement acting on the slope.

Conclusion

GSI rating for slope A and slope B are 38 and 43, respectively. Rock mass properties have been determined and it was not only influenced by GSI rating. Based on kinematic analysis and FEA, both slope A and slope B can be considered as stable. Wire mesh, bolting, weep holes, and surface drainage are needed to prevent water pressure build up and future failure.

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