EARTH SCIENCES
RESEARCH JOURNAL

Earth Sci. Res. J. Vol. 20, No. 2 (June, 2016) : J1 - J9.

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NACIONAL
DE COLOMBIA
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GEOTHECNICS

Classification of cut slopes in weathered meta-sedimentary bedrocks

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ABSTRACT

In order of abundance, the meta-sedimentary rocks along Pos Selim Highway in Perak state Malaysia comprise quartz mica schist, graphitic schist and quartzite layers. Field investigations revealed that these meta-sedimentary rocks have gradational weathering profile based on differences particularly in textures, hardness, lateral changes in colour, and consistency of material extension. The results from uniaxial compressive strength tests confirmed field observations whereby failure occurred mostly on outcrops having joints almost perpendicular to foliation. From the kinematic analyses, the investigated cut slopes are unstable with possibilities of wedge and planar failures. Application of rock mass classification schemes including Rock Quality Designation (RQD) and Rock Mass Rating (RMR) yielded almost similar poor to good quality ranges for each investigated rock mass. While Slope Mass Rating (SMR) classified the cut slopes from stable to unstable slopes, this study categorized them into one actively unstable, four marginally stable and five stable slopes.

Keywords: Weathering, Uniaxial Compressive Strength, Slope stability, Rock Quality Designation, Rock Mass Rating, Slope Mass Rating.

Clasificación de Taludes de Corte en Sustratos Rocosos Metasedimentarios Meteorizados

RESUMEN

En orden de abundancia, las rocas metasedimentarias a lo largo de la carretera Pos Selim, en el estado Perak de Malasia, se componen de esquistos de cuarzo mica, esquistos de grafito y capas de cuarzo. Las investigaciones de campo revelan que estas rocas metasedimentarias tienen perfiles de meteorización progresiva basados en diferencias particulares como textura, dureza, cambios laterales de color y consistencia del material de extensión. Los resultados de los ensayos uniaxiales de esfuerzo de compresión confirmaron las observaciones de campo por las cuales se estableció que las fallas ocurrieron mayormente en los afloramientos con coyunturas perpendiculares hacia la foliación. De los análisis cinemáticos se desprende que los taludes de corte investigados son inestables con posibilidades de fallas planas y de cuña. La utilización de esquemas de clasificación rocosa como el índice RQD (del inglés Rock Quality Designation) y la clasificación geomecánica de Bienawski o RMR (del inglés Rock Mass Rating) evidencia rangos similares de baja y buena calidad para cada masa rocosa estudiada. Mientras que el índice de taludes SMR (del inglés Slope Mass Rating) clasificó los taludes de corte de estables a inestables, este estudio los categorizó de uno activamente inestable, cuatro marginalmente estables y cinco estables.

Palabras clave: Meteorización, esfuerzo de compresión uniaxial, estabilidad de talud, Designación Cualitativa de Roca, Clasificación Geomecánica de Bienawski, índice de SMR.

Record

Manuscript received: 30/08/2015
Accepted for publication: 04/03/2016

How to cite item
Nkpadobi, J. I., Raj, J. K., & Ng, T. T. (2016). Classification of cut slopes in weathered meta-sedimentary bedrocks. *Earth Sciences Research Journal*, 20(2), J1-J9. doi:http://dx.doi.org/10.15446/esrj.v20n2.52781

1. Introduction

Information gathered from field investigations by geologists and engineers is insufficient to predict geotechnical behaviour of rocks and rock masses, which calls for laboratory investigations to ascertain the response of rocks under a wide variety of disturbances. The uniform definition of geotechnical engineering by Murthy (2002), Venkatramaiah (2006), and Das and Sobhan (2013) is that it deals with the application of the principles of soil and rock mechanics to the design of foundations, retaining structures, and earth structures. In this investigated area, previous studies by Mohd Asbi & Associates (2005) and Andrew Malone Ltd (2007) classified the stability of the cut slopes using kinematic analyses. To ensure further understanding of the characterization of the slopes, this study carried out field observations, laboratory and computational analyses of the geotechnical properties of the rocks and then utilized the recorded and derived data for rock mass and slope mass classifications.

2. Geological setting of investigated area

The study area is along Pos Selim Highway in Perak State Malaysia. It traverses through the Titiwangsa Main Range mountainous terrain reaching highest altitude of 1587m above sea level at Gunung Pass and terminating at 1420m above sea level at Perak/Pahang states boarder. This study begins at latitude 4° 33’ 44” and longitude 101° 18’ 86” extending to 17.7km and terminating at latitude 4° 35’ 95” and longitude 101° 20’ 80”. Granite and schist are the two major Formations covering 60% and 40% of the area respectively (Fig. 1). This investigation focused on the schist, which according to Tajul (2003) might probably be of Upper Palaeozoic. In order of abundance the schist consists of quartz mica schist, graphitic schist and quartzite layers. Petrographic analyses by Mohd Asbi & Associates (2005), Andrew Malone Ltd (2007), Nkpadobi (2014) and Nkpadobi et al. (2015) classified the schist as quartz-mica schist, graphitic schist, phyllite and quartzite, and photomicrographs confirm that most of the quartz mica schist are mylonitized.

The major identified failure types in the area include planar sliding, wedge failure, and complex failures characterized by slump and earth flow which occurred along the Gunung Pass axis. The geometrical properties of failed slopes are shown in Table 1. However, the plausible triggering factors for these different types of failures varied. Figs. 2(a) - (c) represent these planar, wedge and complex failures in the area respectively.

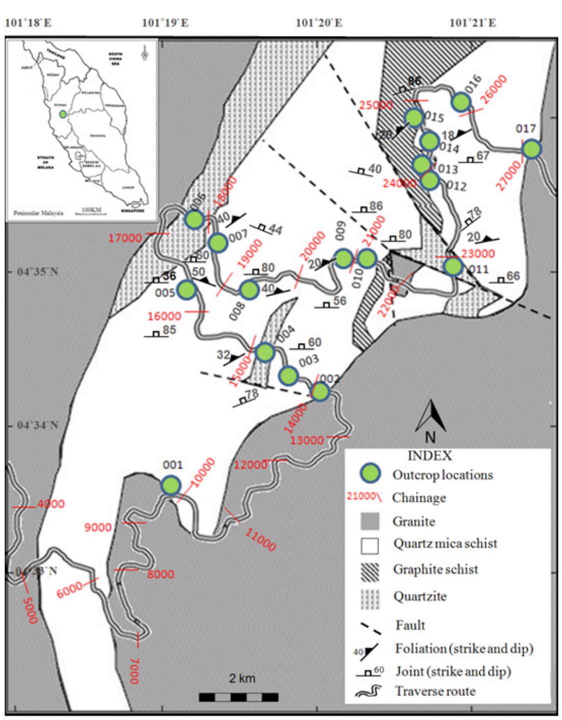

Figure 1. Geological map of the study area.

Table 1. Geometrical properties of failed slopes.

Slope location / Chainage	Slope orientation		Type of failure	Failed zone		Slip angle (Deg)	Slip direction (Deg)
	Slope angle (Deg)	Slope direction (Deg)		Failure height (m)	Lateral extent (m)		
003 / 14350	45	228	Planar	6	15	35	100
004 / 14800	63	236	Wedge	8	18	Various angles	Various directions
006 / 17800	63	110	Planar	6	-	48	100
007 / 18400	63	100	Planar	6	12	50	100
010 / 21200	68	60	Planar	6	-	53	130
014 / 24400	63	250	Complex	40	200	Complex	Complex

Figure 2.
(a) Planar failure. **(b)** Wedge failure. **(c)** Complex failure.

2.1 Description of weathering profile over quartz mica schist

Since the failures in quartz mica schist occurred in the weathered zones, the characteristics of these weathered earth materials were studied in order to interpret the features of the morphological zones. The photograph and schematic diagram of weathering profile of location 010 shown in Fig. 3 is typical of quartz mica schist units. Employing BSI (1981) code of practice for site investigations, the weathering profile was graded. By identifying the lateral changes in colour, texture, hardness, and consistency of material extension, the weathering profiles over the quartz mica schist are broadly differentiated into three zones; pedological soil zone, intermediate zone and bedrock zone corresponding to zones I, II and III respectively. Based on differences particularly in textures and structures of original bedrock as well as degree of preservation of the constituent minerals, these three broad zones are further differentiated into thinner characteristic horizons.

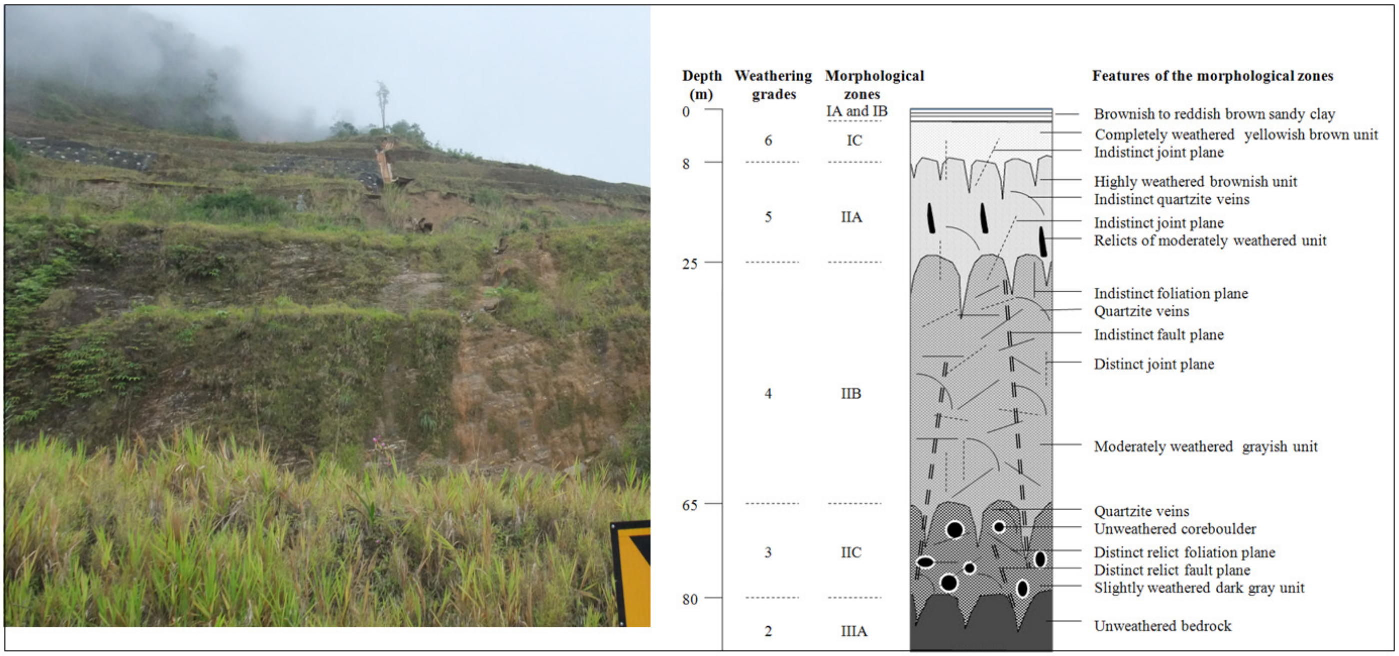

Figure 3. Field photograph and schematic diagram of weathering profile over quartz mica schist at location 010.

Zone I with vertical thickness of less than 8m which corresponds to grade 6 is further sub-divided into IA, IB, and IC . Horizons IA and IB are brownish to reddish brown pedological soil profile, whereby IA which is less than 1m is friable sandy clay, whereas IB which is less than 2m is firm sandy clay. Horizon IC is completely weathered, stiff yellowish brown sandy clay devoid of any distinct discontinuity plane. Greater attention is focused on zone II because of its extensive thickness, complexity, and easy accessibility. With vertical thickness measuring up to 72m, this zone II comprises about 24% vertical thickness of higher horizon IIA (corresponding to grade 5) of highly weathered brownish unit devoid of distinct discontinuity plane, but with intercalations of relicts of moderately weathered units. The lower horizon IIB is about 55% vertical thickness of the entire zone II, comprising moderately weathered grayish coloured quartz mica schist. It corresponds to grade 4, exhibits conspicuous quartzite veins, and distinct joint and foliation planes, but indistinct fault plane. Horizon IIC is the lowest horizon of zone II, and it corresponds to grade 3. With vertical thickness of about 21% of the entire zone II, this slightly weathered dark gray unit comprises distinct relict discontinuity planes and unweathered core-boulders which are very prominent. Zone III which is schematically represented only by morphological zone IIIA is unweathered bedrock which experiences the effect of weathering only along and between structural discontinuity planes.

3. Materials and methodology

3.1 Measurement of rock mass jointing

To ensure adequate representation of the joint parameters, the joint orientations were recorded using scanline and random survey techniques. Joint spacing and persistence were measured with ruler and tape while the aperture was measured with caliper. According to Palmström (2005), joint frequency is defined as the number of joints per meter length, and this was calculated as the inverse of joint spacing. The variations of joint spacing and frequency of joint sets are shown in Table 2. The average volumetric joint count **Jv** was calculated as the sum of the average frequency of the joint sets while field recording of the conditions of the joints include their hydraulic conditions, material infill and roughness. The general characterization of the rock mass jointing for the investigated slopes is presented in Table 3 and these data shall be applied in rock mass and slope mass classifications.

Table 2. Summary of spacing and frequency of individual joint sets.

Cut slopes	Joint sets	Dip dir. /Dip (Deg)	Variation of joint set spacing and frequency				Average spacing (m)	Average frequency
			Min. spacing (m)	Max. spacing (m)	Max. frequency	Min. frequency		
Location 004	J1	192/60	0.02	0.4	50	2.5	0.21	4.76
	J2	182/78	0.1	0.6	10	1.67	0.35	2.86
Location 005	J1	210/60	0.2	0.6	5	1.67	0.4	3.34
	J2	306/36	0.2	0.4	5	2.5	0.3	3.75
	J3	352/85	0.2	0.4	5	2.5	0.3	3.75
	J4	270/46	0.2	0.4	5	2.5	0.3	3.75
Location 006	J1	232/62	0.2	0.6	5	1.67	0.4	3.34
	J2	210/78	0.2	0.6	5	1.67	0.4	3.34
	J3	332/66	0.4	0.6	2	1.67	0.5	1.84
	J4	310/48	0.4	0.6	2	1.67	0.5	1.84
Location 007	J1	260/44	0.02	0.6	50	1.67	0.31	3.23
	J2	328/80	0.02	0.6	50	1.67	0.31	3.23
	J3	322/58	0.02	0.6	50	1.67	0.31	3.23
Location 008	J1	260/74	0.06	0.2	16.67	5	0.13	7.7
	J2	334/80	0.1	0.15	10	6.7	0.13	8.35
	J3	320/64	0.1	0.15	10	6.7	0.13	8.35
Location 009	J1	260/56	0.06	0.2	16.67	5	0.13	10.84
	J2	295/66	0.06	0.4	16.67	2.5	0.23	9.6
Location 010	J1	308/86	0.2	0.6	5	1.67	0.4	2.5
	J2	130/86	0.2	0.6	5	1.67	0.4	2.5
	J3	230/88	0.4	0.6	2.5	1.67	0.4	2.5
	J4	340/80	0.2	0.4	5	2.5	0.3	3.33
	J5	260/66	0.2	0.4	5	2.5	0.3	3.33
Location 011	J1	36/78	0.2	0.6	5	1.67	0.4	3.34
	J2	312/66	0.2	0.4	5	2.5	0.3	3.75
	J3	190/78	0.2	0.4	5	2.5	0.3	3.75
Location 014	J1	145/82	0.2	0.6	5	1.67	0.4	3.34
	J2	180/67	0.2	0.6	5	1.67	0.4	3.34
	J3	240/88	0.2	0.6	5	1.67	0.4	3.34
	J4	20/40	0.3	0.5	3.33	2	0.4	2.65
Location 015	J1	147/80	0.2	0.6	5	1.67	0.4	2.5
	J2	238/86	0.2	0.6	5	1.67	0.4	2.5
	J3	18/38	0.2	0.4	5	2.5	0.3	3.33

Table 3. General characteristics of the rock mass jointing.

Cut slopes	Maximum persistence (m)	Aperture (mm)	Average volumetric joint count **Jv**	Infilling	Hydraulic condition	Condition of discontinuity
Location 004	8	2 - 10	7.62	Clayey	Dripping	Slightly rough surface
Location 005	1	0.25 - 5	14.6	Shrubs and clayey	Damp	Slightly rough surface
Location 006	3	0.1 - 0.25	10.36	None	Damp	Slightly rough surface
Location 007	2	0.25 – 0.5	9.7	Shrubs and clayey	Wet	Slightly rough surface
Location 008	1	0.1 – 0.5	24.4	Clayey	Damp	Slightly rough surface
Location 009	0.8	0.1 – 0.25	20.44	Clayey	Damp	Slightly rough surface
Location 010	0.6	0.1 – 0.25	14.16	None	Dry	Slightly rough surface
Location 011	1	0.1 – 0.25	10.84	Shrubs	Flowing	Slightly rough surface
Location 014	0.6	0.1 – 0.25	12.67	None	Dry	Slightly rough surface
Location 015	1	0.1 – 0.25	8.33	None	Dry	Smooth

3.2 Kinematic stability assessment

Kinematic stability assessment method was applied in order to identify critical planes of weakness in the rock slopes and outline potential danger and likely modes of failure. Measurements of discontinuities were carried out with the use of both scanline and random survey techniques. The slope face orientation was also measured. These analyses were based on Markland's test as described in Hoek and Bray (1981). Pole intensity greater than 4% was regarded as a major discontinuity and assumed friction angle along the discontinuities in the rocks is averaged at 25°. Table 4 shows the summary of the orientations of the major discontinuity planes and slope face orientations.

Table 4. Summary of the orientations of slope locations and major discontinuities.

Summary of the investigated discontinuities								Slope face orientation	
Cut slopes	Elevation (m)	Discontinuity data	Dip (Deg)	Dip Direction (Deg)	Discontinuity Set	Intensity	Remark	Slope angle (Deg)	Slope direction (Deg)
Location 004	1003	115	32	110	J1	>8%	Foliation	63	236
			60	192	J2	>8%	Joint		
			78	182	J3	>4%	Joint		
Location 005	1061	120	50	98	J1	>8%	Foliation	63	246
			60	210	J2	>8%	Joint		
			36	306	J3	>4%	Joint		
			85	352	J4	>4%	Joint		
			46	270	J5	>4%	Joint		
Location 006	1114	119	40	100	J1	>8%	Foliation	63	110
			62	232	J2	>8%	Joint		
			78	210	J3	>8%	Joint		
			66	332	J4	>4%	Joint		
			48	310	J5	>4%	Joint		
Location 007	1129	120	40	100	J1	>8%	Foliation	63	100
			44	260	J2	>4%	Joint		
			80	328	J3	>4%	Joint		
			58	322	J4	>4%	Joint		
Location 008	1155	110	40	102	J1	>8%	Foliation	63	98
			74	260	J2	>8%	Joint		
			80	334	J3	>4%	Joint		
			64	320	J4	>4%	Joint		
Location 009	1206	105	56	260	J1	>8%	Joint	63	250
			20	190	J2	>4%	Foliation		
			66	295	J3	>4%	Joint		
Location 010	1214	117	86	308	J1	>8%	Joint	68	60
			86	130	J2	>8%	Joint		
			88	230	J3	>4%	Joint		
			80	340	J4	>4%	Joint		
			66	260	J5	>4%	Joint		
Location 011	1275	120	20	228	J1	>8%	Foliation	70	250
			78	36	J2	>8%	Joint		
			66	312	J3	>4%	Joint		
			78	190	J4	>4%	Joint		
Location 014	1348	102	82	145	J1	>8%	Joint	63	250
			18	335	J2	>8%	Foliation		
			67	180	J3	>8%	Joint		
			88	240	J4	>8%	Joint		
			40	20	J5	>4%	Joint		
Location 015	1366	101	80	147	J1	>8%	Joint	70	250
			20	337	J2	>8%	Foliation		
			86	238	J3	>8%	Joint		
			38	18	J4	>4%	Joint		

3.3 Uniaxial compressive strength test

Very large rock blocks were collected in the field with use of sledge hammer and chisel for this very test, and the laboratory determination of the strength of the rock samples was carried out using ISRM (1981a) standard. As shown in Fig. 4(a), the rock samples were prepared into right circular cylinders ranging from 12.5 to 15cm in height and 5cm in diameter using diamond embedded rock core drilling machine while Fig. 4(b) shows the set up of this test. The uniaxial compressive strength of the samples was then determined under dry condition which is their ultimate strength.

Figure 4.
(a) Some of the rock core samples. (b) Set up of uniaxial compressive strength test.

4. Results and discussions

4.1 Uniaxial compressive strength (UCS)

It was observed that the tested samples failed axially following joint trends and fissures, almost perpendicular to foliation. This confirmed field observations whereby failure occurred mostly on outcrops having joints almost perpendicular to foliation. Although some metamorphic rocks tested by Horino and Ellickson (1970) and Broch (1974) showed that uniaxial compressive strength of the rocks are almost the same at different orientations of foliation, but in these tested quartz mica schist samples, there are significant differences in the strength in different orientations of the well developed foliations. Where the orientation of the foliation to the horizontal is less than 45° in some quartz mica schist samples, failure occurred along the foliation plane. Several tests showed consistency of failure modes in graphitic schist, whereby majority of the failures occurred along orientations of foliations at 45° with minimal oblique joint trends. The determined uniaxial compressive strength of quartz mica schist samples shown in Table 5 ranges from 56MPa to 117MPa, while graphitic schist has value of 87MPa. According to classification of the uniaxial compressive strength of rocks by Deere and Miller (1966), Bieniawski (1978), ISRM (1978 and 1981b), the rocks of the study area fall within medium to high strength. This gives insight into the observed failure mechanisms whereby most of the failures were controlled by discontinuities.

Table 5. Uniaxial compressive strength (UCS) and Rock Quality Designation (RQD).

Cut slopes location number	Chainage	Rock type	UCS (MPa)	Jv	RQD (%)	Rock quality
004	14800	Quartz mica schist	110	7.62	90	Good
005	16400	Quartz mica schist	117	14.6	66.8	Fair
006	17800	Quartz mica schist	76	10.36	80.8	Good
007	18400	Quartz mica schist	89	9.7	83	Good
008	19300	Quartz mica schist	84	24.4	34.5	Poor
009	20900	Quartz mica schist	56	20.44	47.6	Poor
010	21200	Quartz mica schist	61	14.16	68.3	Fair
011	22750	Quartz mica schist	97	10.84	79.2	Good
014	24400	Quartz mica schist	76	12.67	73.2	Fair
015	24780	Graphitic schist	87	8.33	87.5	Good

4.2 Rock mass classification

Rock mass classification has been developing over the years and various classification schemes have considered a lot of factors such as water content, discontinuities and rock strength. In this study, the considered rock mass classification schemes include Rock Quality Designation (RQD) and Rock Mass Rating (RMR).

4.2.1 Rock Quality Designation (RQD)

Rock Quality Designation (RQD) index was developed by Deere (1963). It is defined as the percentage of intact rock mass length that are 10cm or longer from borehole drill cores. In absence of drill core logs but where discontinuity traces are visible in rock surface exposures, Palmström (1982) suggested that Rock Quality Designation (RQD) might be estimated from the number of discontinuities per unit volume using below equation:

$$RQD = 115-3.3Jv \tag{1}$$

Where **Jv** known as the volumetric joint count is the sum of the number of joins per unit length for all joint sets. Using already determined values of **Jv** and applying equation 1, the RQD values for the rock masses are presented in Table 5. According to Deere (1968) relationship between RQD and the engineering quality of rock mass, the RQD values determined in this work range from 34.5% to 90% covering a broad range of poor, fair and good quality rocks. In view of non-consideration of joint orientation, joint condition, type of joint, infilling and stress condition by RQD index, Singh and Goel (1999) advised that its sole consideration for classification is insufficient to provide adequate description of a rock mass.

4.2.2 Rock Mass Rating (RMR)

Rock Mass Rating (RMR) or Geomechanics classification was initially developed by Bieniawski (1976). The system has evolved due to a better understanding of the importance of the different parameters and increased experience leading to changes to the ratings of parameters. As a result of these advancements, the Bieniawski (1989) version was employed in this study. This scheme uses six parameters: Uniaxial compressive strength of rock material, rock quality designation (RQD), spacing of discontinuities, condition of discontinuities, groundwater conditions and orientation of discontinuities. As explained in Bieniawski (1989), estimation of RMR is the sum of the total ratings of each of the above listed six parameters. Considering the dip angles of the joints in these high cut slopes, -5 rating adjustment for discontinuity orientations was used. As presented in Table 6, the discontinuity condition is the sum of joint persistence, aperture, roughness, infilling and weathering ratings. This RMR scheme has five rock mass classes determined from total ratings ranging from very poor rock to very good rock.

In order to determine the RMR for the investigated extended cut slope, the already determined values of the six parameters were substituted with their individual ratings and the rock masses range from fair to good rocks. The estimation of the RMR of the investigated rock masses are presented in Table 7. Unlike in RQD index where quartz mica schist at location 004 yielded highest value of 90%, in this RMR only quartz mica schist at locations 010 and 014 and graphitic schist at location 015 yielded highest value of 62 and designated good rocks. Although the remaining cut slopes are in quartz mica schist and designated fair rocks, the quartz mica schist at locations 008 and 009 maintained the lowest RMR values of 43 and 44 respectively. The advantage of rock mass classification using RMR is that it incorporates geological, geometric and engineering parameters in arriving at a quantitative value of the rock mass quality.

Table 6. Determination of condition of discontinuities.

Cut slopes locations / Chainage	Persistence		Aperture		Roughness		Infilling		Weathering range		Conditions of discontinuities
	Max. value (m)	Rating	Range (mm)	Rating	Condition	Rating	Condition	Rating	Condition	Rating	
004 / 14800	8	2	2-10	2	Slightly rough	3	Soft filling <5 mm	2	Slightly weathered	5	14
005 / 16400	1	4	0.25-5	1	Slightly rough	3	Soft filling <5 mm	2	Slightly weathered	5	15
006 / 17800	3	4	0.1-0.25	4	Slightly rough	3	None	6	Moderately weathered	3	20
007 / 18400	2	4	0.25-0.5	1	Slightly rough	3	Soft filling <5 mm	2	Moderately weathered	3	13
008 / 19300	1	4	0.1-0.5	1	Slightly rough	3	Soft filling <5 mm	2	Slightly weathered	5	15
009 / 20900	0.8	6	0.1-0.25	4	Slightly rough	3	Soft filling <5 mm	2	Highly weathered	1	16
010 / 21200	0.6	6	0.1-0.25	4	Slightly rough	3	None	6	Moderately weathered	3	22
011 / 22750	1	4	0.1-0.25	4	Slightly rough	3	Soft filling <5 mm	2	Slightly weathered	5	18
014 / 24400	0.6	6	0.1-0.25	4	Slightly rough	3	None	6	Moderately weathered	3	22
015 / 24780	1	4	0.1-0.25	4	Smooth	1	None	6	Moderately weathered	3	18

Table 7. Rock Mass Rating (RMR) of investigated rock masses.

Cut slopes locations / Chainage	Rock type	UCS rating	RQD rating	Spacing of discontinuities rating	Condition of discontinuities rating	Groundwater condition ratings	Adjustment for discontinuities orientation	RMR	Rock mass classes
004 / 14800	Quartz mica schist	12	20	10	14	4	-5	55	Fair rock
005 / 16400	Quartz mica schist	12	13	10	15	10	-5	55	Fair rock
006 / 17800	Quartz mica schist	7	17	10	20	10	-5	59	Fair rock
007 / 18400	Quartz mica schist	7	17	10	13	7	-5	49	Fair rock
008 / 19300	Quartz mica schist	7	8	8	15	10	-5	43	Fair rock
009 / 20900	Quartz mica schist	7	8	8	16	10	-5	44	Fair rock
010 / 21200	Quartz mica schist	7	13	10	22	15	-5	62	Good rock
011 / 22750	Quartz mica schist	7	17	10	18	0	-5	47	Fair rock
014 / 24400	Quartz mica schist	7	13	10	22	15	-5	62	Good rock
015 / 24780	Graphitic schist	7	17	10	18	15	-5	62	Good rock

4.3 Slope stability analysis of the rock cuts

The kinematic analyses of the discontinuity sets revealed the possibility of both wedge and planar failures. At location 004, analysis shows intersection of J2 and J3 along 267˚/27˚ line orientation. Four intersections within the critical zone were recorded at location 005; J2 and J3 intersected along 280˚/34˚, J3 and J4 intersected along 265˚/30˚, J2 and J4 intersected along 266˚/46˚, and J3 and J5 intersected along 136˚/54˚ line orientations. Location 006 recorded three intersections; J1 and J4 intersected along 49˚/29˚, J1 and J3 intersected along 128˚/37˚, while J1 and J2 intersected along 155˚/27˚. At location 007, only one intersection between J1 and J3 along 53˚/31˚ was recorded. The same single intersection was also recorded at location 008 between J1 and J3 along 58˚/32˚. J1 and J3 intersected along 242˚/54˚ at location 009. There was also only one intersection between J2 and J4 at location 010 along 48˚/65˚. Only J3 and J4 intersected along 262˚/56˚ at location 011, while J1 and J3 intersected along 222˚/61˚ at location 014. Analysis on the graphitic schist at location 015 yielded only one intersection between J3 and J4 having orientation of 326˚/28˚ line of intersection. As shown in Fig. 5(a), there are possibilities of only wedge failures at locations 004, 010, 011, 014 and 015, while Fig. 5(b) shows possibilities of wedge and planar failures at locations 005, 006, 007, 008 and 009. Based on the kinematic analysis of these discontinuity sets, it is anticipated that all cut slopes in this investigated area are unstable.

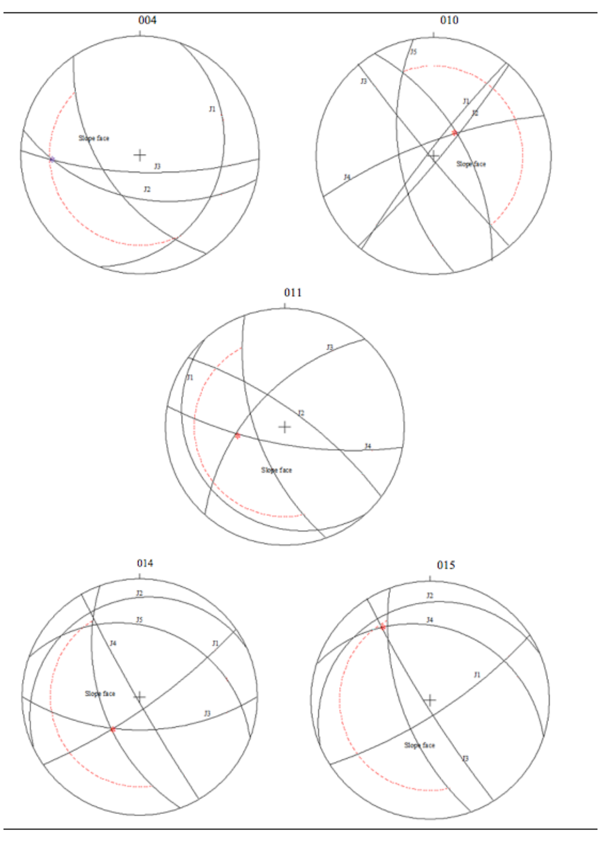

Figure 5a. Stereographic projections showing possibilities of wedge failures.

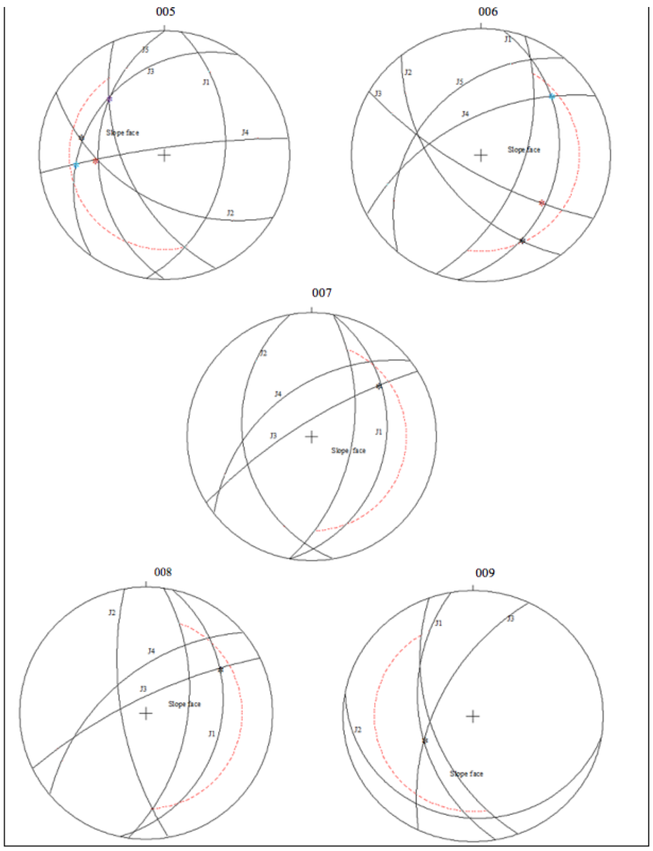

Figure 5b. Stereographic projections showing possibilities of wedge and planar failures.

4.4 Classification of the rock cut slopes

It may be useful to visualize slopes as existing in one of the following three stages:

i. Stable - The margin of stability is sufficiently high to withstand all destabilizing forces.

ii. Marginally stable - Likely to fail at some time in response to destabilizing forces reaching a certain level of activity.

ii. Actively unstable - Slopes where destabilizing forces produce continuous or intermittent movements.

Slope Mass Rating (SMR) proposed by Romana (1985) is a method to access the stability of both natural and cut slopes. According to Romana (1993) and Romana et al. (2003), SMR is obtained from RMR of Bieniawski (1989) as shown in equation 2 by adding a factorial adjustment factor which depends on the relative orientation of joint and slope and by adding another factor depending on the method of slope excavation. Rating adjustment for discontinuity orientations in RMR is not considered and this basic RMR is designated RMR_b.

$$SMR = RMR_b + (F_1.F_2.F_3) + F_4 \quad (2)$$

Whereby F_1 ranges from 1.0 to 1.5 and depends on the parallelism between discontinuity and it fits into the relationship:

$$F_1 = (1\text{-}\sin A)^2 \quad (3)$$

and A = the angle between the dip directions of the slope and joint.

F_2 depends on the joint dip angle (βj).

For toppling failure, this parameter maintains 1.0 value and F2 thereby fits into the relationship:

$$F_2 = (\tan \beta j)^2 \quad (4)$$

F3 depends on the relationship between dips of slope (βs) and joint (βj).

F4 is a correction factor that depends on the excavation method used and it is fixed empirically.

In Table 8, P represents plane failure, αs represents slope dip direction, while αj represents joint dip direction and T represents toppling failure. As proposed by Romana (1993), the adjustment ratings of these factors are presented in Table 8 while the classes of SMR are presented in Table 9. The SMR was carried out using the independent joint sets from each investigated cut slope and finally applying equation 2. By analyzing the joint sets as shown in Table 10, SMR classified the stability of slopes as completely stable, stable, partially stable, unstable and completely stable. Using these five categories of SMR stability by Romana (1993), this study categorized the slopes as existing in three stages: Stable, marginally stable and actively unstable as presented in Table 10.

Table 8. Adjustment rating of F_1, F_2, F_3 for joints and method of slope excavation (Romana, 1993).

Joint Orientation	Very favourable	Favourable	Fair	Unfavourable	Very unfavourable
P │αj - αs│	>30	30~20	20~10	10~5	<5
T │(αj – αs) - 180│	>30	30~20	20~10	10~5	<5
F_1 (for P & T)	0.15	0.40	0.70	0.85	1.00
P │βj│	<20	20~30	30~35	35~45	>45
F_2 (for P)	0.15	0.40	0.70	0.85	1.00
F_2 (for T)	1.00	1.00	1.00	1.00	1.00
P βj – βs	>10	10~0	0	0~-10	<-10
T βj + βs	<110	110~120	>120	--	--
F_3 (for P & T)	0	-6	-25	-50	-60
Method	Natural slope	Presplitting	Smooth blasting	Blasting/Ripping	Deficient blasting
F_4	+15	+10	+8	0	-8

Table 9. Classification of slope according to SMR (Romana, 1993).

SMR	Class	Description	Stability	Failure	Support
81~100	I	Very good	Completely stable	None	None
61~80	II	Good	Stable	Some blocks	Spot
41~60	III	Fair	Partially stable	Some joints or many wedges	Systematic
21~40	IV	Poor	Unstable	Planar or large wedges	Important / Corrective
0~20	V	Very poor	Completely unstable	Large wedges or circular failure	Re-excavation

Table 10. Slope Mass Rating of the investigated cut slopes.

Cut slopes	RMRb	Slope dip dir. /Dip (Deg)	Joint sets	Joint dip dir. /Dip (Deg)	F1	F2	F3	F4	SMR	SMR Stability	Stability by this study
Location 004	60	236/63	J1	192/60	0.15	1	-50	+8	61	Stable	Stable
			J2	182/78	0.15	1	0		68	Stable	
Location 005	60	246/63	J1	210/60	0.15	1	-50	+8	61	Stable	Marginally stable
			J2	306/36	0.15	0.85	-60		60	Partially stable	
			J3	352/85	0.15	1	0		68	Stable	
			J4	270/46	0.4	1	-60		44	Partially stable	
Location 006	64	110/63	J1	232/62	0.15	1	-50	+8	65	Stable	Stable
			J2	210/78	0.15	1	0		72	Stable	
			J3	332/66	0.15	1	-6		71	Stable	
			J4	310/48	0.15	1	-60		63	Stable	
Location 007	54	100/63	J1	260/44	0.15	0.85	-60	+8	54	Partially stable	Marginally stable
			J2	328/80	0.15	1	0		62	Stable	
			J3	322/58	0.15	1	-50		55	Partially stable	
Location 008	48	98/63	J1	260/74	0.15	1	0	+8	56	Partially stable	Marginally stable
			J2	334/80	0.15	1	0		56	Partially stable	
			J3	320/64	0.15	1	-6		55	Partially stable	
Location 009	49	250/63	J1	260/56	0.7	1	-50	+8	22	Unstable	Actively unstable
			J2	295/66	0.15	1	-6		56	Partially stable	
Location 010	67	60/68	J1	308/86	0.15	1	0	+8	75	Stable	Stable
			J2	130/86	0.15	1	0		75	Stable	
			J3	230/88	0.15	1	0		75	Stable	
			J4	340/80	0.15	1	0		75	Stable	
			J5	260/66	0.15	1	-50		68	Stable	
Location 011	52	250/70	J1	36/78	0.15	1	-6	+8	59	Partially stable	Marginally stable
			J2	312/66	0.15	1	-50		53	Partially stable	
			J3	190/78	0.15	1	-6		59	Partially stable	
Location 014	67	250/63	J1	145/82	0.15	1	0	+8	75	Stable	Stable
			J2	180/67	0.15	1	-6		74	Stable	
			J3	240/88	0.7	1	0		75	Stable	
			J4	20/40	0.15	0.85	-60		67	Stable	
Location 015	67	250/70	J1	147/80	0.15	1	-6	+8	74	Stable	Stable
			J2	238/86	0.7	1	0		75	Stable	
			J3	18/38	0.15	0.85	-60		67	Stable	

5. Conclusion

The field and laboratory tests, computation of results and schematic representations were aimed at classifying the stability of the investigated cut slopes. From the uniaxial compressive strength tests, it was observed that the combination of joints and foliations induced structurally controlled failures in the tested samples. RQD and RMR schemes utilized for rock mass classification yielded almost similar poor to good quality ranges for each investigated rock mass. While the RQD values range from 34.5% to 90% covering a broad range of poor, fair and good quality rocks, RMR yielded values from 43 to 62 which cover only fair and good quality rocks. RMR applied in this study is very useful as a tool for the preliminary assessment of slope stability whereby it classified seven of the slopes cut into fair rocks and only three slopes cut into good rocks. Although based on the kinematic analyses carried out in this study, it is anticipated that the entire cut slopes are unstable with possibilities of wedge and planar failures, SMR gave comparative stability classes among these cut slopes, categorizing them actively unstable, marginally stable and stable slopes. Considering the determined unstable slopes, there is need for the use of expensive and wider in-situ and laboratory testing equipment in order to reasonably predict the stability of these cut slopes before any further infrastructural development in the area is carried out.

Acknowledgement

Special thanks to University of Malaya for sponsoring this research through Postgraduate Research Fund, grant number: PS362/2010B.

References

Andrew Malone Ltd. (2007). Landslide study at Ch 23+800 Simpang PulaiLojing Highway, Malaysia. Report to minister of works of Malaysia.

Bieniawski, Z. T. (1976). Rock mass classification in rock engineering. In: Z. T. Bieniawski eds. Proceedings of the symposium on Exploration for Rock Engineering. Cape Town: Balkema. 1, 97-106.

Bieniawski, Z.T. (1978). Rock mass rating systems in engineering practice. Symposium on Rock Classification Systems for Engineering Purposes, ASTM, STP; 984: 17-34.

Bieniawski, Z. T. (1989). Engineering rock mass classifications. Wiley and Sons, New York, 251 pp.

Broch, E. (1974). The influence of water on some rock properties. Proceedings of the Third Congress of the International Society for Rock Mechanics, Themes 1-2: Advances in Rock Mechanics, Reports of Current Research. 2(A), 33-38.

BSI (British Standards Institution) (1981). Code of practice for site investigations. BS 5930:1981, BSI London.

Das, B. M. & Sobhan, K. (2013.) Principles of geotechnical engineering. 8th Edition. Cengage Learning, Stamford, CT 06902, USA, 726 pp.

Deere, D. U. (1963). Technical description of rock cores for engineering purposes. Rock Mechanics and Rock Engineering, 1, 16-22.

Deere, D. U. (1968). Geological considerations. In: Rock Mechanics In: Engineering Practice. R.G. Stage and D.C. Zienkiewicz eds. Wiley. New York, 1-20.

Deere, D. U. & Miller R.P. (1966). Engineering classification and index properties of rock. Technical Report No. AFNL-TR-65-116. Albuquerque, NM: Air Force Weapons Laboratory.

Hoek, E. & Bray, J. W. (1981). Rock slope engineering. The Institute of Mining and Metallurgy, London, England, 358 pp.

Horino, F. G. & Ellickson, M. L. (1970). A method of estimating strength of rock containing planes of weakness. US Bureau of Mines Report of Investigation 7449, 26 pp.

ISRM (International Society for Rock Mechanics) (1978). Standardization of laboratory and field tests. Int. Jour. Rock Mech and Min. sci. and Geomech. Abst. 15, 348 pp.

ISRM (International Society for Rock Mechanics) (1981a). Suggested methods for rock characterization testing and monitoring. Brown E. T.(ed), Oxford Pergamon Press, 211 pp.

ISRM (International Society for Rock Mechanics) (1981b). Basic technical description of rock masses. International Journal of Rock Mechanics and Mining Sciences and Geomechanics Abstracts, 18, 85-110.

Mohd Asbi & Associates Sdn Bhd. (2005). Surface investigation report on proposed cut slope between Ch 23+000 and Ch 24+560 Simpang Pulai – Gua Musang – Kuala Berang package 2, Malaysia. Report to Ministry of works Malaysia.

Murthy, V. N. S. (2002). Geotechnical Engineering, Principles and Practices of Soil Mechanics and Foundation Engineering. Marcel Dekker, New York, 1029 pp.

Nkpadobi , J. I. (2014). Geotechnical properties of meta-sedimentary bedrock and the stability of cut slopes along the Pos Selim highway in Perak State, Malaysia, PhD Thesis, University of Malaya, Kuala Lumpur, Malaysia.

Nkpadobi, J. I., Raj J. K. & Ng T. F. (2015). Failure mechanisms in weathered meta-sedimentary rocks. Environmental Earth Sciences. Springer Berlin Heidelberg. 73 (8), 4405-4418.

Palmström, A. (1982). The volumetric joint count - a useful and simple measure of the degree of rock jointing. Proceedings of the 4th congress on International Association Engineering Geology., Delhi. 5, 221-228.

Palmström, A. (2005). Measurements of and Correlations between block size and Rock Quality Designation (RQD). Tunnelling and Underground Space Technology, 20, 362-377.

Romana, M. R. (1985). New adjustment ratings for application of Bieniawski classification to slopes. Proceedings of International Symposium on Role of Rock Mechanics in Excavations for Mining and Civil Works. ISRM, Mexico City, 59-68.

Romana, M. R. (1993). A geomechanical classification for slopes: Slope mass rating. In: Comprehensive Rock Engineering, v. 3, ch. 23 (Hudson, J. D., ed.) Pergamon Press, Oxford, 575-599.

Romana, M., Serón, J. B. & Montalar, E. (2003). SMR Geomechanics classification: Application, experience and validation ISRM–Technology roadmap for rock mechanics. South African Institute of Mining and Metallurgy, 1-4.

Singh, B. & Goel, R. K. (1999). Rock mass classification: A practical approach in civil engineering. Elsevier science. 282 pp.

Tajul, A. J. (2003). Engineering geological assessment and slope failures along the Pos Selim Cameron Highland Highway. Seminar Penyelidikan Jangka Pendek (Vot F), 11th-12th March 2003, University of Malaya.

Venkatramaiah, C. (2006). Geotechnical engineering. Revised Edition. New Age International, 926 pp.