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A STUDY ON LANDSLIDE POTENTIALS AND SLOPE STABILITY OF THE ROAD AT ZWEKABIN RANGE NEAR KHALAUKNOS VILLAGE, HPA-AN TOWNSHIP

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Abstract

The research aims to mention the landslide hazard and mitigation measures in the Zwekabin Range. This study road is located at the eastern part of the Zwekabin Range near Khalauknos Village, Hpa-an Township, Kayin State. This area is mainly composed of cherty limestone and lime-mudstone of Moulmein Limestone (Permian in age). Moreover, the area lies in the Zwekabin-Belin Fault Zone, a right-lateral strike-slip fault. Hilly terrain with steep slopes is found in the most of places at the study route and structurally unstable. The big landslide occurred on 15th, July in 2018 on the road at Zwekabin Range. Rock mass characterization and slope stability analysis are carried out for potential of landslides in the research area to know causes of landslides and preventing systems. In the study, although the behaviors of the rocks are not quite different along the road, the strength and engineering properties of the rocks are varied from place to place. Plane failures and wedge failures commonly occur. Most of the slopes are dangerous attaining stability because some rocks are very weak in geological condition as well as slope condition. According to the slope mass rating (SMR) analysis, most of the slopes are completely unstable and unstable slope conditions. In addition, all slopes have not been supported by suitable retaining structure and systematic drained system. Therefore, preventing systems and controlling techniques should be made to mitigate the causes of landslide.

Keywords: Landslide, plane failure, wedge failure, slope stability, slope mass rating

Introduction

The present study mainly aims to conduct the analysis of the landslide hazards along the road at the base of the Zwekabin Hill near Naunglon (east) village, Hpa-an Township, Kayin State. The geological and geotechnical parameters of the rocks along the road are mainly studied and measured in this research. This area is mainly composed of carbonate sedimentary rocks. The behaviors of the rocks are not quite different along the car-road but not the same. Moreover, the strength and engineering properties of the rocks are varied from place to place because of the structural controls. The big landslide occurs on 15th, July in 2018 at this road.

Location

This area lies between north latitudes 16º 46' 00" to 16º 50' 00" and east longitudes 97º 39' 00" to 97º 43' 00", covering UTM map No. 1697-09. The research road sector along the road is about 1200 m long near Naunglon (east) village, Hpa-an Township, Kayin State which is shown in Figure (1).

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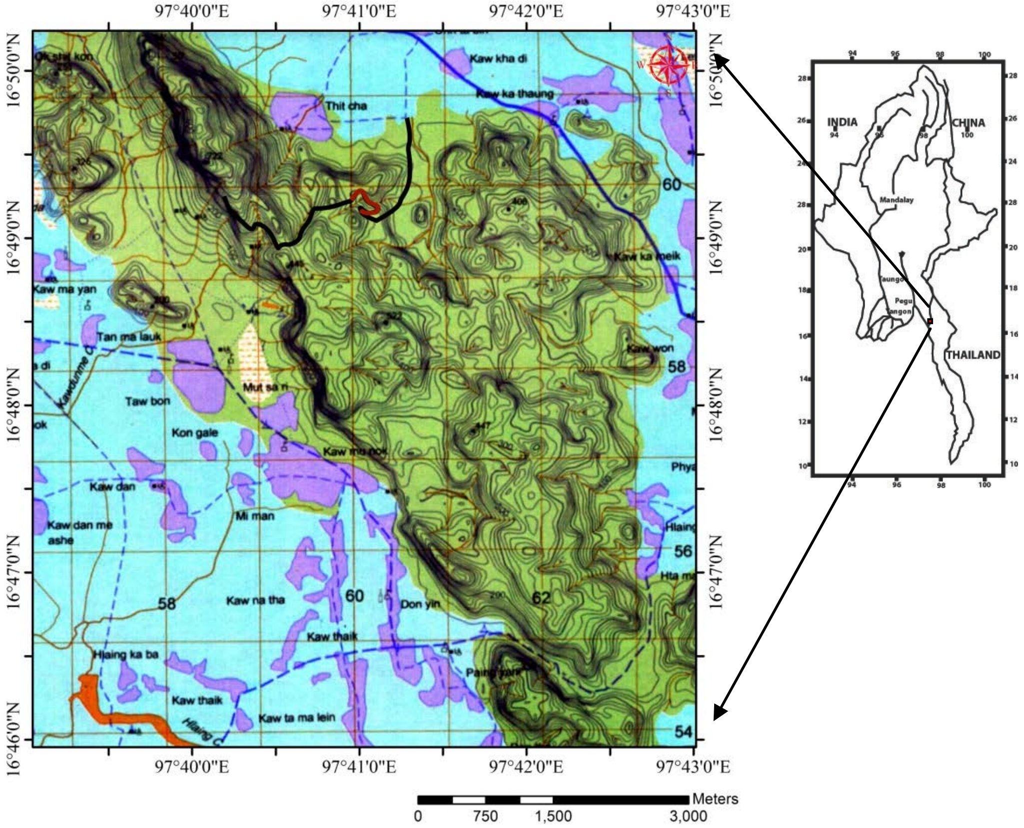


Figure 1 Location map of the research area

Physiography

Physiographically, Zwekabin Range is highly elevated and rugged terrain which is composed of limestone over the whole range (Figure 2). The elevation was divided into four divisions based on their elevations where green colored area shows flat plain and its elevation is less than 300 m. Although the elevations were classified into four, the other three want to designate as highly elevated and rugged terrain where elevation has greater than 300 m because the whole lithology is same. The average elevation is 400 m in this area and the highest peak has at the top of the Zwekabin Range which is elevated about 722 m above sealevel.

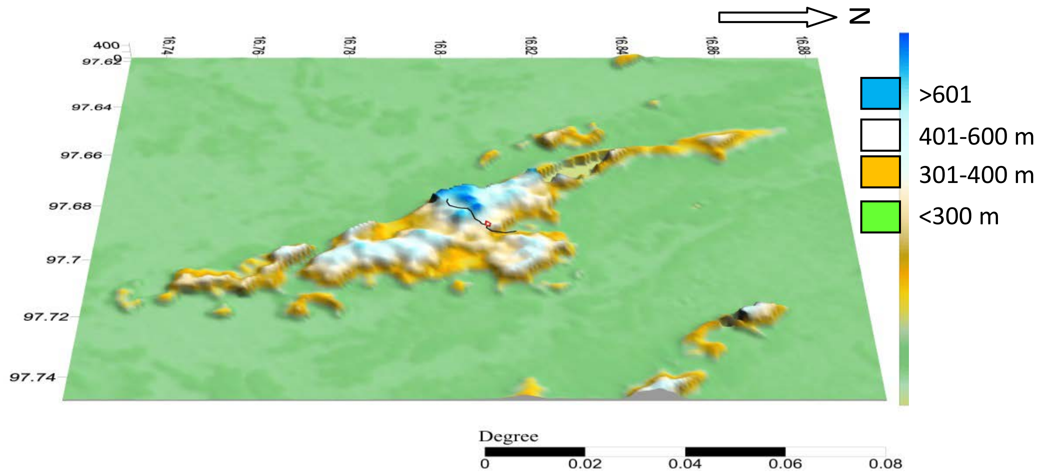


Figure 2 Physiographic map of the research area

Drainage Pattern

In the research area, the centripetal pattern and sub-dendritic pattern are mainly evolved at the central and western part of the research area (Figure 3). The whole area of Zwekabin Range is covered by the centripetal pattern which is reflected by the underlying Moulmein Limestone. Sub-dendritic pattern with fine to medium texture is found at the north-western part of the study area where is covered by the underlying Taungnyo Formation.

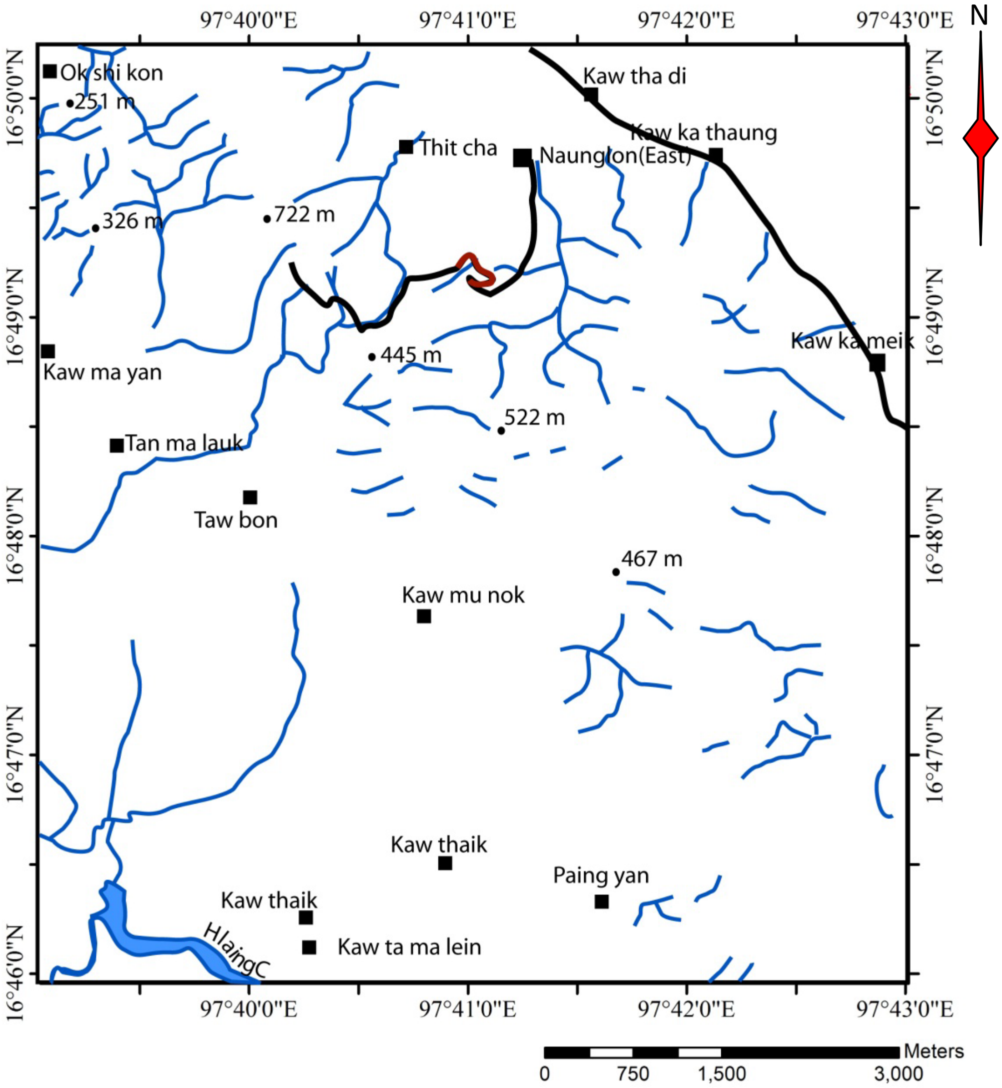


Figure 3 Drainage pattern of the research area

Climate and Rainfall

The climatic condition of the study area is tropical. Rainfall made the rock units as highly weather nature, so rock fall block as risk effect. According the comparative study of rainfall data by using histogram, highly rainfall shows in June, July and August in 2015-2018. The significant rainfall data are highest in July and August, 2018 (Table-1 & Figure 4).

Table 1 Monthly rainfall data of the research area

	Jan	Feb	Mar	April	May	June	July	Aug	Sept	Oct	Nov	Dec
2015	2.24	0	1.42	1.86	14.05	30.52	43.92	35.71	20.77	10.82	1.56	0
2016	0.86	0	0	0.08	11.07	30.25	29.7	36.22	27.51	11.97	0.24	0
2017	2.12	0	0	4.61	12.63	24.93	43.02	28.64	18.00	9.31	0	0.63
2018	0	0	0	1.77	12.91	32.8	62.4	53.00	19.24	12.1	0	0

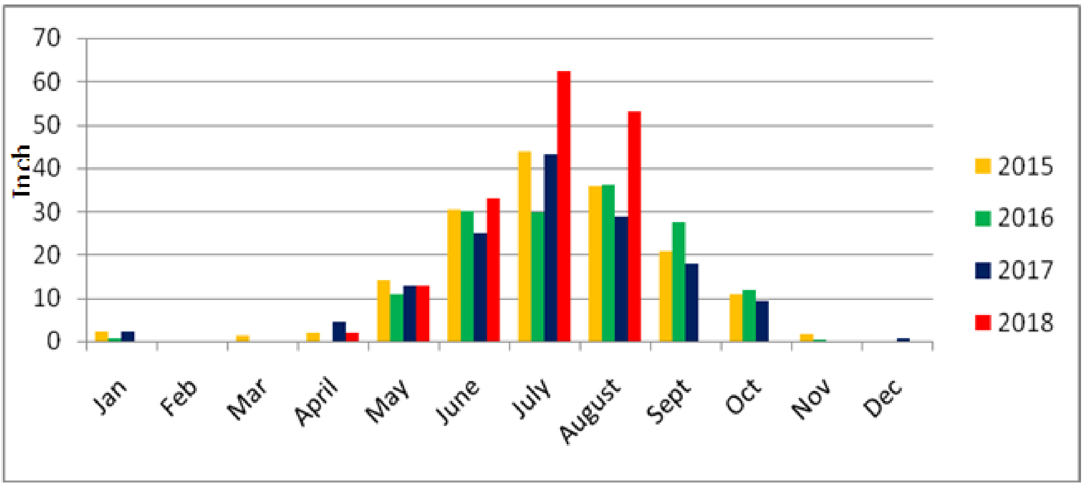


Figure 4 Histrogram shows comparative study of monthly rainfall data

Regional Geologic Setting

The rocks of Taungnyo Formation (Carboniferous to Early Permian), Moulmein Limestone (Middle to Late Permian) and Alluvium (Quaternary) covered in the study area with different relief which is shown in Figure (13).

The rocks of the Taungnyo Formation are exposed at the northern part of the Zwekabin Range and southern part of the Hpa-an Town. The rocks are mainly composed of clastic units; thin bedded, whitish grey to pinkish colored siltstone intercalated with thinnly laminated shale, partly fine grained nodular sandstone which is shown in Figure (5).

Moulmein Limestone is mostly composed at the Zwekabin Range with gentle dipping. The other isolated hills with karst topography are also composed of Moulmein Limestone. The rocks are consist of medium to thick bedded, grey to dark grey colored lime-mudstone and brecciated limestone of the Moulmein Limestone (Figures 6-8).

Most of the flatplain are covered by reddish brown to yellowish brown colored, thick alluvial soils.

Geological Structures

The research area is mainly characterized by NNW-SSE trending stratigraphic units in eastern part and NW-SE trending in central and western part. Besides, the major longitudinal fault with normal sense occurred at the western flank of the Zwekabin Range which is trending nearly north-south in direction (Figure 9 & Figure 10). Another thrust fault is also trending parallel with the above normal fault (Figure. 11). Besides, the Zwekabin (east) Fault is trending nearly NW-SE direction at the east part of the Zwekabin range which occurred as the right-lateral strike-slip faults (Figure 12). Moreover, the anticlinal fold is occurred with NNW-SSE trending at the western part of the Zwekabin Range.

Figure 5 Thin bedded, whitish grey to pinkish colored siltstone intercalated with thinnly laminated shale, partly fine grained nodular sandstone of Taungnyo Formation at the northern part of Zwekabin Range

Figure 6 Medium to thick bedded, grey to dark grey colored lime-mudstone of the Moulmein Limestone exposed at the west of Naunglon (East) Village

Figure 7 High pressure radial joint pattern in lime-mudstone with chert nodule at the Naunglon (east) village

Figure 8 Intersection of two normal faults with vertical striation (N16° 57' 00.2", E97°25'51.0", Facing - 260°)

Figure 9 Normal fault scarp at the western part of the Zwekabin Range

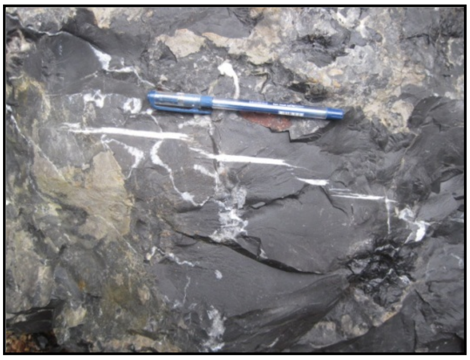
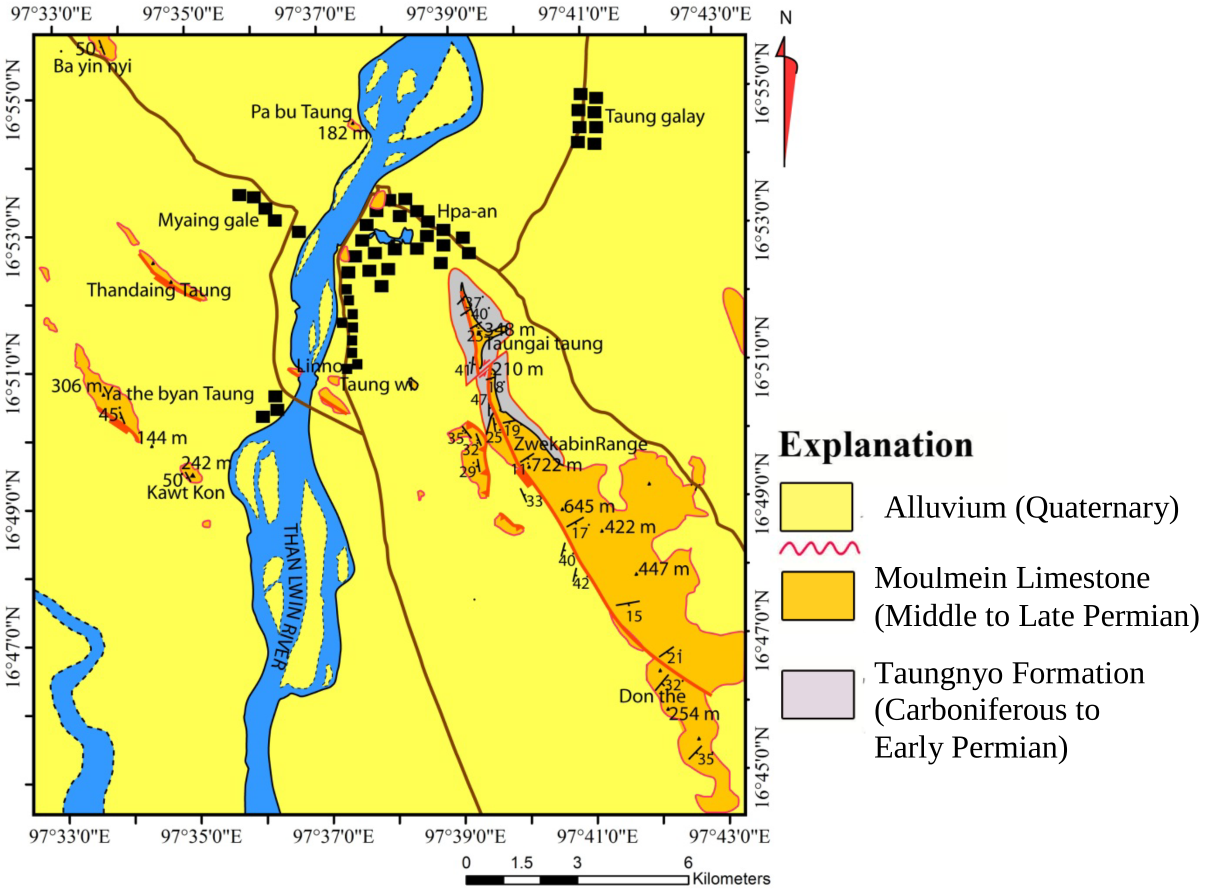


Figure 10 Extensional normal fault planes cutting in calcite vein at the west of Naunglon (East) Village

Figure 11 Thrust sheet at the west of the Zwekabin Range

Figure 12 Strike-slip fault at the east side of the Zwekabin Range



Geology by Aung Kyaw Myat and Aung May Than (2018)

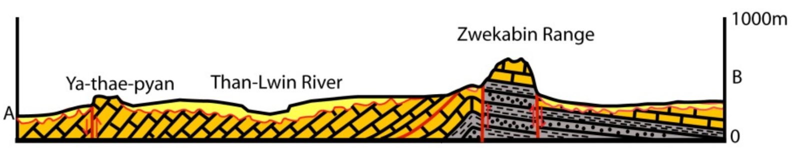


Figure 13 Geological map of the research area

Past Landslides

Annually landslide happens always occur at hilly region of Hpa-an area in rainy season. But many minor landslides are found in this rainy season because of the significant rainfall effect. Most of the slope failures are formed by the following effects; (a) uncontrolled cutting the slope, (b) coincide with fault line (c) highly rainfall effect and (d) weathering effect. According to the Varnes (1978), there are many types of landslide along the car-road such as rockfall (Figures 14 & 16), debris flow (Figure 17, 18, 19 and 20), creep (15) and size of the landslide is shown in Figure (21).

Figure 14 Rockfall in lime-mudstone of Moulmein Limestone (N16°49'13.1, "E97° 41'11.4", Facing - 170°)

Figure 15 Creep in cherty limestone of Moulmein Limestone (N16°49' 14.6"E97°41'13.6", Facing - 80°)

Figure 16 Rockfall in cherty limestone of Moulmein Limestone (N16°49'14.6"E97° 41'14.3", Facing - 340°)

Figure 17 Debris flow in brecciated limestone of Moulmein Limestone (N16°49'15.4"E97°41'09.6", Facing - 280°)

Figure 18 Rockfall in brecciated limestone of Moulmein Limestone (N16°49'15.4" E97°41'09.6", Facing - 290°)

Figure 19 Debris flow in brecciated limestone of Moulmein Limestone (N16°57'00.2", E97°25'51.0", Facing - 280°)

Figure 20 Big debris flow in the research area (N16°57'00.2", E97°25'51.0")

Figure 21 Size of the debris flow on google map

The big landslide is triggered on 15th July, 2018 because of the significant highly rainfall in Kayin State. The width of the landslide scar or debris flow area along the road is 45 meter width and 25 meter length until the base of the mountain. This landslide badly destroyed the road where can't be used in future if the systematic retaining structure and drainage don't use.

Methodology

Landslide classification adopted for this research is that proposed by Cruden and Varnes (1978). The rock mass rating (RMR) was computed according to Bieniawski (1989), with adding rating values for five parameters: i) strength of intact rock, ii) RQD, iii) spacing of discontinuities, iv) condition of discontinuities, and v) water inflow through discontinuities and/ or pore pressure ratio.

The volumetric joint count (Jv) has been described by Palmström (1982). It can be measured from the joint set spacings within a volume of rock mass as Jv =1/S1+ 1/S2+ 1/S3+…+Nr/5 where S1, S2, S3 are the joint set spacings and also random joints can be included by assuming a random spacing (Nr) for each of these where Nr = the number of random joints.

The strength of the rock mass is measured by using point load test of laboratory tests. The volumetric joint count was used to measure by using the formula of Jv = 1/S1+1/S2+1/S3+---+Nr/5, where Nr means the numbers of random joints to determine the RQD (Romana, 1993). It is not possible to obtain good correlations between RQD and Jv. In 1982, the formular RQD = 115 - 3.3 Jv was applied, which was presented by Palmström (1982).

Spacing of discontinuities is the distance between them, measured along a line perpendicular to discontinuity planes. Determining the condition of discontinuities is not simple parameter, as it includes several parameters such as (i) roughness, (ii) separation, (iii) filling material, (iv) persistence and (v) weathering of walls. The groundwater which accounts for the influence of the water pressure, with particular reference to the underground excavation are classified either; completely dry, damp, wet and dripping or flowing.

Geomechanics classification of Rock Mass Rating (RMR), Bieniawski (1989) system has been used to find Slope Mass Rating (SMR) (Romana, 1993). The “Slope Mass Rating” (SMR) is obtained from RMR by adding a factorial adjustment factor; depending on the relative orientation of joints and slopes and another adjustment factor depending on the method of rock slope excavation. Field observations and measurements of discontinuities by using this formula SMR = RMR+ (F1 x F2 x F3) + F4 are the main method for finding the SMR.

The stereographic method of kinematic analysis (Goodman, 1998) and (Jeongi-gi and Kulatilake, 2001) is mostly useful for assessment of the stability of discontinuity planes. Slope orientation, discontinuity sets orientation and friction angle of the each Formation in the research area are used for kinematic analysis as the three main parameters. Data of the strike and dip values of slopes and discontinuities have been obtained from a discontinuity survey and pole plot, respectively. The slope face is shown as a great circle and the friction angle is represented by an interior circle.

Results and Discussions

A rock mass is composed of a system of rock blocks and fragments separated by discontinuities forming a material in which all elements behave in mutual dependence as a unit. The main features constituting a rock mass are large variations in the composition and structure of rocks as well as in the properties and occurrence of the discontinuities intersecting the rock that lead to a complicated composition and structure of the rock mass.

The following rock mass tests are measured to know the properties or characteristics of the rocks that are observed by field observations, descriptions and indirect test and supported by laboratory test also (Figure 22).

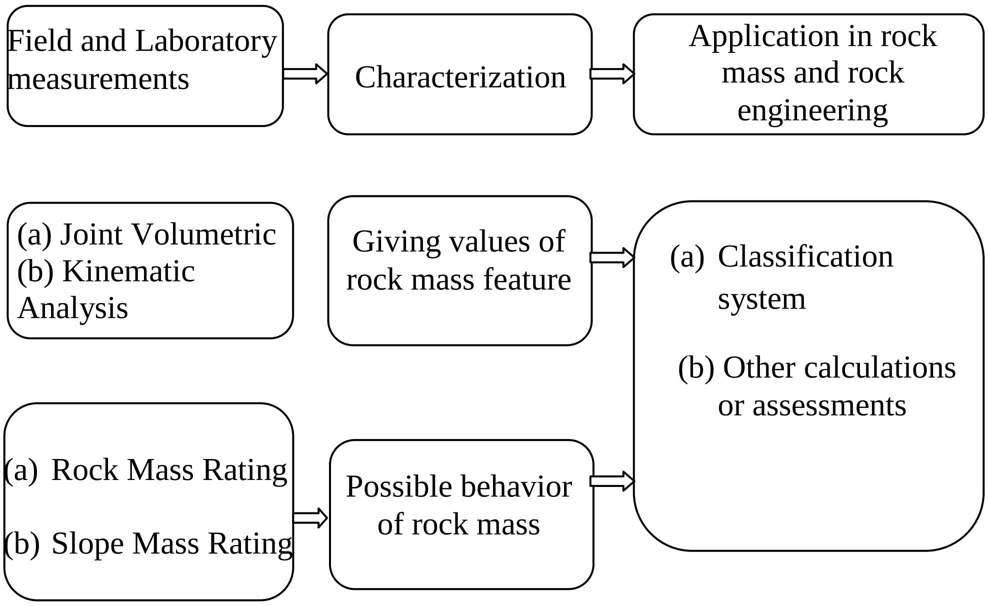


Figure 22 Idealized rock mass analysis diagram for present study

The Characteristic of Discontinuities

Joint Volumetric

The volumetric joint count of the area is a measure measured from the joint set spacings within a volume of rock mass and also random joints can be included by assuming a random spacing (Table 2). It is defined as number of joints per m³. The slope site localities map is shown in Figure (23).

Figure 23 Slope site localities map of the research area

Table 2 Joints volumetric counts data in this area

Site No.	Location	Rock Unit	Length (m)	Joint set spacing (m)				(Jv)
				1/J1	1/J2	1/J3	Nr/5	
1	N16°49'13.1" E97°41'11.4"	Lime-mudstone with chert nodules	1	10.25	5.32	4.21	5.67	25.45
2	N16°49'16.4" E97°41'11.4"	Lime-mudstone with chert nodules	1	12.34	7.63	3.23	5.89	29.09
3	N16°49'16.1" E97°41'13.4"	Cherty Limestone	1	14.8	10.23	4.32	2.47	31.82
4	N16°49'14.6" E97°41'13.6"	Cherty Limestone	1	13.3	8.31	2.33	5.15	29.09
5	N16°49'14.6" E97°41'14.3"	Cherty Limestone	1	14.11	9.26	3.21	3.72	30.30
6	N16°49'14.6" E97°41'15.5"	Lime-mudstone	1	13.12	9.43	4.21	2.63	29.39
7	N16°49'15.1" E97°41'09.5"	Lime-mudstone	1	9.21	7.34	4.55	3.14	24.24
8	N16°49'15.4" E97°41'09.6"	Lime-mudstone	1	14.14	9.21	3.23	3.72	30.30
9	N16°49'15.7" E97°41'10.2"	Brecciated limestone	1	12.4	6.34	2.22	4.49	25.45
10	N16°57'00.2" E97°25'51.0"	Brecciated limestone	1	15.5	11.45	3.21	2.57	32.73

Geotechnical Analysis

The geotechnical analyses are made on the basis of rock mass rating and slope mass rating which is shown in Table (3).

Rock Mass Rating

Bieniawaski's Geomechanics Classification system provides a general rock mass rating (RMR) increasing with rock quality from 0 to 100. It is based upon five universal parameters: strength of the rock, drill core quality, groundwater conditions, joint and fracture spacing, and joint characteristics.

Table 3 Rock mass rating measuring data in the field

Site No.	Strength	RQD	Joint spacing	Joint condition	Groundwater condition	RMR	Rock Mass Class
1	7	31	5	9	4	56	Fair rock
2	4	19	5	9	7	44	Fair rock
3	4	10	5	15	4	36	Poor rock
4	7	19	5	9	7	47	Fair rock
5	4	15	5	9	4	33	Poor rock
6	4	18	5	15	7	44	Fair rock
7	4	35	5	9	4	58	Fair rock
8	4	15	5	9	4	33	Poor rock
9	7	31	5	9	4	56	Fair rock
10	4	7	5	9	4	29	Poor rock

According to this classification, Sites 3, 5, 8 and 10 are highly jointed and highly weathered rocks that composed of poor rocks as rock mass rating. The conditions of these sites can danger for slope stability if these slope sites don't use the systematic slope treatment. Moreover, the rocks composed of sites 3, 5, 8 and 10 have poor strength according to the rating measured by point load test. Most of the other rock units are moderately jointed and fair rocks.

Slope Mass Rating

The Slope Mass Rating (SMR) is obtained from RMR by adding a factorial adjustment factor; depending on the relative orientation of joints and slopes and another adjustment factor depending on the method of rock slope excavation. The formula of SMR is RMR+ (F1 x F2 x F3) + F4, where F1 depends on parallelism between joints and slope face strike, F2 refers to joint dip angle in the planar mode of failure, F3 reflects the relationship between slope face and joints dip angles and F4 refers to the adjustment factor for the method of excavation which has been fixed empirically which is illustrated in Table (4).

Slope mass rating addresses both planar sliding and toppling failure modes, no additional consideration is made for sliding on multiple joint planes. The description of the SMR Class is provided in Table (5).

Table 4 Adjustment rating for joint (after Romana, 1993)

Case	Very Favorable	Favorable	Fair	Unfavorable	Very unfavorable
P $a_j - a_s$ T $a_j - a_s - 180°$ P/T $F_1 = (1 - \sin a_j - a_s)^2$	$> 30°$ 0.15	$30° - 20°$ 0.4	$20° - 10°$ 0.7	$10° - 5°$ 0.85	$< 5°$ 1.00
P B_j P $F_2 = \tan^2 B_j$ T F_2	$< 20°$ 0.15 1.00	$20° - 30°$ 0.4 1.00	$30° - 35°$ 0.7 1.00	$35° - 45°$ 0.85 1.00	$> 45°$ 1.00 1.00
P $B_j - B_s$ T $B_j - B_s$ P/T F_3	$> 10°$ $< 110°$ 0	$10° - 0°$ $110° - 120°$ -6	$0°$ $> 120°$ -25	$0° - (-10°)$ – -50	$< -10°$ – -60
F_4 Adjusting factor for excavation method	Natural slope + 15	Pre-splitting + 10	Smooth blasting + 8	Blasting or mechanical 0	Deficient blasting − 8

P- Planar failure a_s- Slope dip direction a_j- Defect dip direction
T- Toppling failure B_s- Slope dip B_j- Defect dip

Table 5 SMR Classes defined by Romana (1993)

Class	SMR	Description	Stability	Failures	Support
I	81 – 100	Very good	Completely stable	None	None
II	61 – 80	Good	Stable	Some blocks	Occasional
III	41 – 60	Normal	Partially stable	Some joints or many wedges	Systematic
IV	21 – 40	Bad	Unstable	Planner or big wedges	Importance/ Corrective
V	0 – 20	Very bad	Completely unstable	Big planner or soil like	Re-excavation

A total of 10 sites have been selected covering the transportation route for slope mass rating assessment and the results and the rock types are described in Table (6).

Table 6 Slope mass rating measuring data in the field

Site No.	RMR	F1	F2	F3	F4	SMR	SMR Class	Stability
1	56	0.85	1	5	-6	56.85	III	Partially stable
2	44	0.85	1	5	-6	44.85	III	Partially stable
3	36	0.15	1	5	-25	17.15	V	Completely unstable
4	47	0.85	1	5	-6	47.85	III	Partially stable
5	33	0.15	1	5	-25	14.15	V	Completely unstable
6	44	0.85	1	5	-25	25.85	IV	Unstable
7	58	0.85	1	5	-25	39.85	IV	Unstable
8	33	0.15	1	5	-25	14.15	V	Completely unstable
9	56	0.85	1	5	-25	37.85	IV	Unstable
10	29	0.15	1	5	-25	10.15	V	Completely unstable

The SMR values of the 10 sites ranges from Class III to Class V. The SMR Class IV and Class V sites hold significant threat to the commuters as well as the road infrastructure.

Sites 3, 5, 8 and 10 are assessed as completely unstable (Class V) and these sites are the most dangerous for slope stabilization as SMR assessment. Sites 6, 7 and 9 are identified as unstable (Class IV) according to slope mass rating. From field observation, the Class IV sites involve smaller rock fragments having larger volume than the SMR Class III Sites. The last sites 1, 2 and 3 are assessed as partially stable (Class III) and these sites are possible joint related failures.

Kinematic Analysis

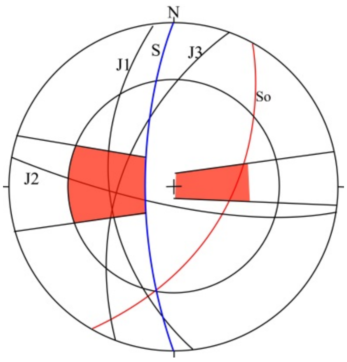
The conventional kinematic analyses for sliding along discontinuities involve constructing the great circles of joints, slope surface, faults, bedding plane, etc. and interpreting the location of intersections. In the kinematic analysis, the relationship between individual great circle and slope inclination is important to assess the slope stability. According to the stereographic projection study, primary and secondary critical possible failure zones can be identified on the stereograph. If the primary critical possible failure zone is in front of the slope face where the cutting by discontinuities planes is inclined less than the slope face, then wedge and plane failures are possible. If the primary critical possible failure zone is in the back side of the slope face, there can be a possible toppling failure. Besides, the side of the primary critical possible failure zone of the slope face is described as the secondary critical possible failure zone.

According to kinematic analyses, Goodman (1998), and Jeongi and Kulatilake (2001), the following determinations have been made based on the relationship of discontinuities, slope faces and friction angle (ϕ) obtained from RMR values by using the stereographic projection as shown in Table (7) and Figure (24).

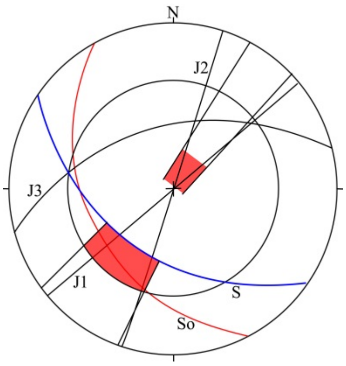
Table 7 Parameters and Results of kinematic analysis of the study area

Site	Site slopes parameters			Discontinuity/ bedding plane or intersection	Mode of Failure	Failure category
	Slope angle	Slope Direction	Frictional angle (ϕ)			
1	70	270	35	So,J1,J2,J3	P,W	Potential
2	46	215	35	So,J1,J2	P,W,T	Potential
3	50	275	25	J3	P	Potential
4	63	220	35	So,J1,J2	P	Potential
5	60	65	25	B,J1,J3	P, T	Potential
6	65	50	35	J1,J2	P	Potential
7	65	50	35	J1,J2	P	Potential
8	70	20	25	B,J1,J2,J3	W,T	Potential
9	69	255	35	J1,J2	W,P	Potential
10	75	265	35	So, J2,J3	W,P	Potential

Where W=Wedge Failure, P = Plane Failure, T = Toppling Failure, S_0 = Bedding planes, S= Slope face, J = Joint plane, F = Fault plane



Site (1)



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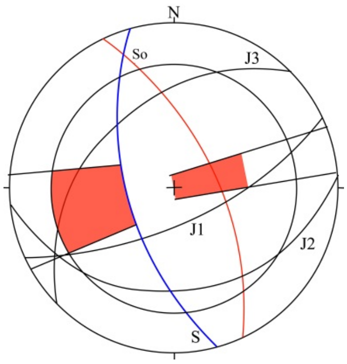

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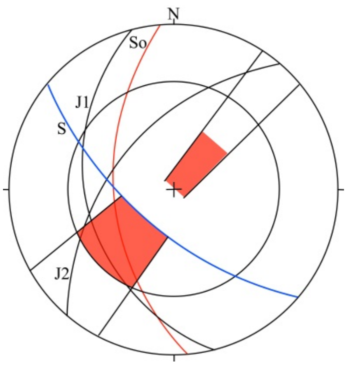
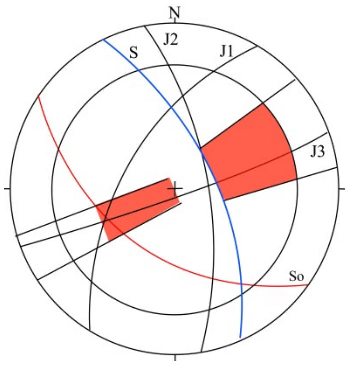
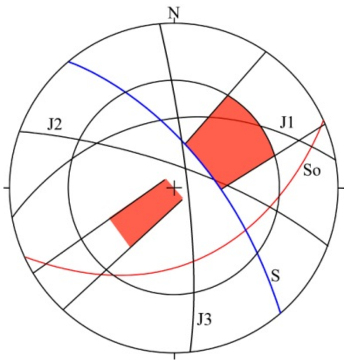


Site (4)



Site (5)



Site (6)

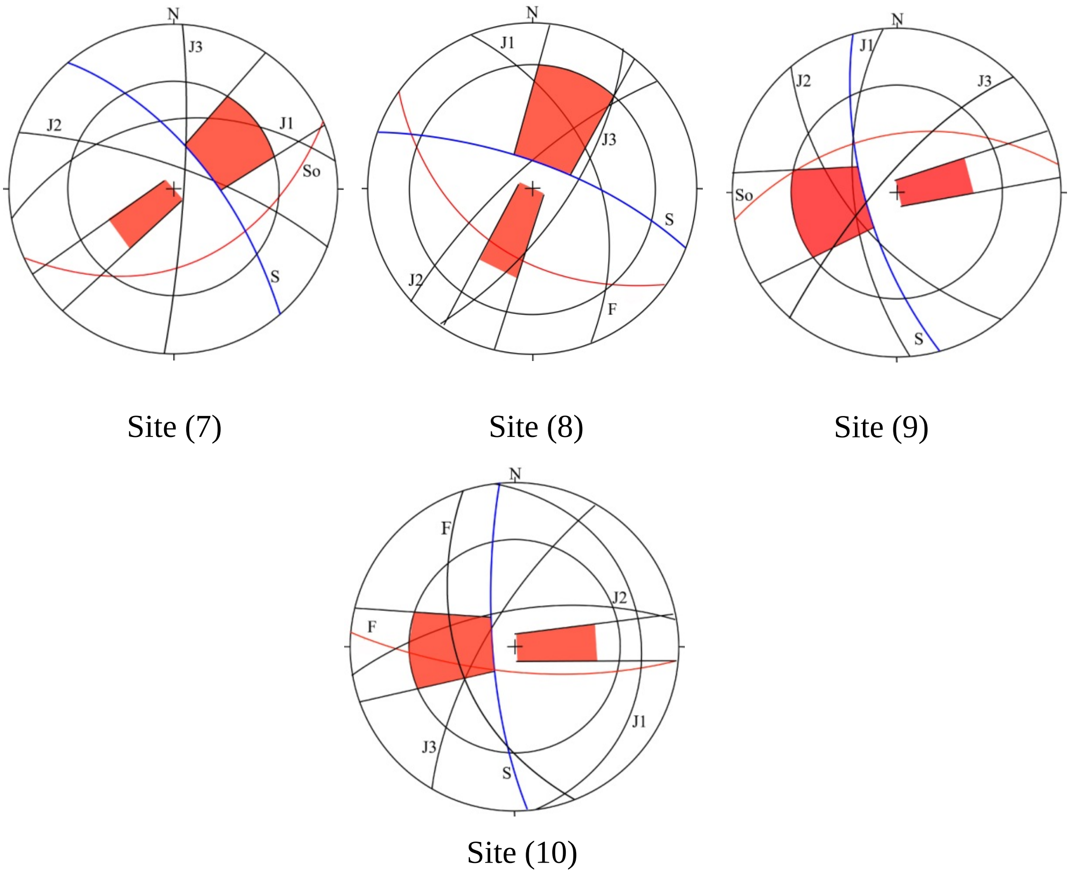


Figure 24 Kinematic analysis of sites (1) to (10) on the relationship of the discontinuities and slope face parameters

Accordingly, most of the slopes possibly have failures and the most prominent failures are wedge failure and planar failure, but some places can have toppling failure. Besides, all these slopes are lack systematic retaining structures and drainpipe.

Conclusions and Recommendations

The present study area is situated along the Car-road, at the base of the Zwekabin Hill near Khalauknos village, Hpa-an Township, Kayin State. This area is mainly composed of carbonate sedimentary rocks especially Moulmein Limestone (Middle to Late Permian) and Alluvium (Quaternary) covered in the study area.

The research area is mainly characterized by NNW-SSE trending stratigraphic units in eastern part and NW-SE trending in central and western part. Besides, the major longitudinal faults with north-south in direction occurred at the eastern and western flank of the Zwekabin Range.

There are many types of landslide along the car-road such as rockfall, debris flow and creep. The big landslide is found on 15th July, 2018 because of the significant rainfall effect which is one of the debris flows.

Moreover, Slope 3, 5, 8 and 10 are highly jointed and highly weathered rocks that composed of poor rocks as rock mass rating. Moreover, the SMR values of the 10 sites have from

Class III to Class V. Sites 3, 5, 8 and 10 are assessed as completely unstable (Class V) and Sites 6, 7 and 9 are identified as unstable (Class IV). These sites are very dangerous to use for transportation. According to the kinematic analysis, most of the slopes possibly have failures and the most prominent failures are wedge failure and planar failure. Besides, the strength and joint condition of rock mass as well as slope condition are also influenced in forming landslide along the road.

It can be recommended that to prevent the slope failures, these slopes should be reduced the slope inclination depends on the orientation and density of discontinuities, planes and mode of failures with systematic retaining structures and drainage pipe. Remedial and mitigation measures seem to be inadequate during the construction of this road. Additional risk assessment study is recommended for prioritizing the necessary mitigation measures and safety of the travellers.

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