Kinematic Analysis of Noklak Landslide, Tuensang District, Nagaland

Mademshila Jamir,* C Nokendangba Chang,* Temsulemba Walling* and Keneiravinuo*

Abstract

A very complex landslide has plagued Noklak town in the eastern part of Nagaland for more than two decades. The magnitude of the landslide has grown both in its influence and coverage area of 1.84 sq. km in last few years affecting the only route connecting the International Trade Centre at Pangsha (Dan), uprooting many households, cultivated areas and also posing a threat to nearly one-fifth of the town population presently. It is incorporated in SOI toposheet no. 83 N/4 and lies in 95°00'39" E longitudes and 26°11'52" N latitudes. The present study aims to identify the causative factors of this land instability by employing the method of kinematic analysis of the slope material to determine the potential mode of failure. These analyses were performed from 1,195 joint attitudes collected from in-situ rock exposures in the field to determine the dominant joints that control the instability in the area. The strength of the rocks was calculated by Point Load Test data on 50 rock samples. Both Rock Mass Rating (RMR) and PLT value indicate poor rock quality and low values for the rocks. SMR (Slope Mass Rating) values for this slope fall in Class IV and results from the kinematic analysis shows both planar and wedge type of failure indicating several micro-slips within the study area and absence of firm bedding.

Keywords: Noklak; Nagaland, Kinematic Analysis; RMR; SMR; Landslides

Introduction

Nagaland, a north-eastern state in the Indian sub-continent is infested with landslides because of intense tectonic activities causing slope instabilities. Noklak town is situated in the eastern part of Nagaland that is characterized by rugged topography of moderately

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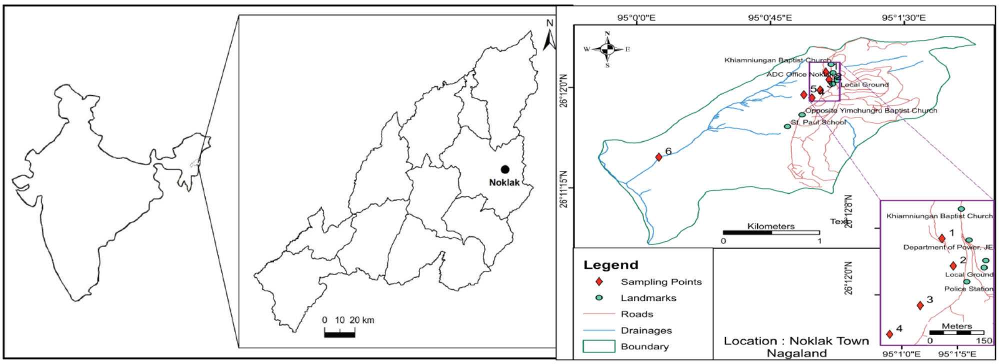
dissected structural hills and valleys with high drainage incisions and is located at 95°00’39” east longitudes and 26°11’52” north latitudes which are incorporated in Survey of India (SOI) topographic sheet no. 83 N/4. A minor land instability which initially started in 1980 at the south-western part of the town has aggravated into a major landslide affecting 1.84 sq. km. The area is occupied by weak lithology and with an average slope of about 50°, it is highly unstable and susceptible to slope failures (Fig. 1).

The present study investigates the landslide that was reactivated in 2004 and subsequent failure in the succeeding years. The main objectives of the present study are to ascertain the influence of geo-mechanical properties of rock on the slope instability, to determine the possible mode of failure and to develop appropriate mitigation measures.

Fig. 1: Map of the study area



MATERIALS AND METHODS

Rock Mass Rating (RMR)

The Rock Mass Rating, a geomechanical classification system for rocks developed by Z.T. Beinawski in 1972 and 1973considers various geological parameters that influence rock instability and represent them with one overall comprehensive index of rock mass quality.

The five parameters on the basis of which rocks are classified using the RMR system are:

- Uniaxial Compressive Strength of rocks (UCS): A strength characteristic of rock for evaluating rock mass classification and analyze slope instability (Thurro, 1997).
- Rock Quality Designation (RQD): Developed by Deere 1967, to provide a quantitative estimate of rock mass quality from drill core logs. RQD is defined as the percentage of intact core pieces longer than 100mm in the total length of the core.

- The spacing of discontinuities: The stability of rock slopes is significantly influenced by the structural discontinuity in the rock in which the slope is excavated. Persistence of discontinuities defines, together with spacing, the size of blocks that can slide from the face.
- Condition of discontinuities: Roughness of discontinuity surface such as joints, is the measure of the inherent unevenness and waviness of the surface of discontinuity relative to its mean plane.
- Groundwater conditions: This accounts for the influence of the water pressure, with particular reference to the underground excavation. It can be classified as dry, damp, wet and flowing (Bieniawski, 1989).

The strength parameters of the rocks are measured using the Point Load Testing machine (PLT) or Schmidt hammer. To analyse the rock samples collected from the field, the point load test is opted for. The point load strength index I_S is calculated using the following relation:

$$I_S = P / De^2$$

Where, P = pressure obtained at failure, De = equivalent diameter of the rock sample

In the case of point load strength index less than 1 MPa, Uniaxial Compressive Strength test (UCS) is applied. This low value may be the outcome of weak slope material influenced by the presence of water. Hence for shale, which is the dominant rock type in the affected area, the UCS equation (Singh et al, 2013) is given as

UCS = 14.4(PLI) (PLI = Point Load Index)

In the case of the use of Schmidt hammer for determining the compressive strength of intact rocks, the UCS equation employed (Deere and Miller, 1966) is given as

$$\text{UCS} = 6.9 \times 10^{(0.16 + 0.0087\, R_n \rho)}$$

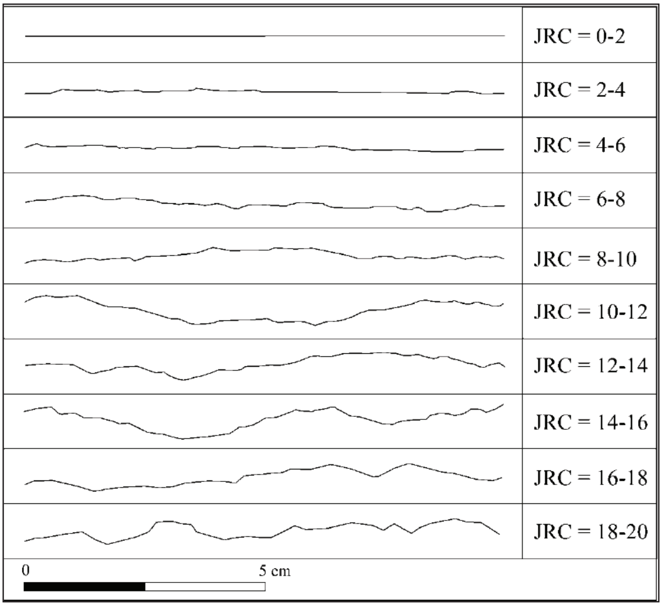
Where, R_n = Schmidt hammer rebound number, ρ = Rock density

Palmstrom (1982) estimated RQD from the number of joints per volume given by the following equation:

$$\text{RQD} = 115 - 3.3\, J_v,$$

Where J_v = the sum of the number of joints per unit length for all joint sets, known as the volumetric joint count. The condition of the joint is inferred from the inherent surface smoothness or unevenness and waviness relative to the plane of the joint. Joint roughness can be felt by touch and is recognized in the field as very rough, rough, slightly rough, smooth, polished and slickensided surfaces. The JRC is estimated (Table 1) by comparing the appearance of a discontinuity surface with a standard profile (Barton et al., 1977).

Table 1: Joint roughness profiles with JRC values (Barton and Choubey, 1977)



Groundwater conditions are made by visual observations and accordingly their ratings are estimated. The algebraic sum of these five parameters gives the RMR values for a slope.

The values of all the five parameters are then entered in the rock mass rating system as shown in table 2. Rock mass classes determined from total ratings are given in Table 3.

Table 2: Rock Mass Rating System (after Bieniawski, 1989)

Sl. No	Parameter		Range of values						
1	Strength of intact rock material (MN/m^{-2})	Point load strength index	>10 MPa	4-10 MPa	2-4 MPa	1-2 MPa	For this low range- uniaxial compressive test is preferred		
		Uniaxial compressive strength	>200 MPa	100-200 MPa	50-100 MPa	25-50 MPa	5-25 MPa	1-5 MPa	<1 MPa
	Rating		15	12	7	4	2	1	0
2	Drill core quality RQD		90-100%	75-90%	50-75%	25-50%	<25%		
	Rating		20	17	13	8	3		
3	Spacing of joints		>2 m	0.6-2 m	200-600 mm	60-200 mm	<60 mm		
	Rating		20	15	10	8	5		

4	Condition of joints		Very rough surface Not continuous No separation. Unweathered wall rock	Slightly rough surfaces Separation < 1 mm; Slightly weathered walls	Slightly rough surface Separation < 1 mm Highly weathered walls	Slickensided surfaces or Gouge <5 mm thick or Separaion 1-5 mm continuous	Soft gouge <5 mm or Separation >5 mm Continuous
	Rating		30	25	20	10	0
5	Groundwater	Inflow per 10m tunnel length (l/m)	None	<10	10-25	5-125	>125
		(jointwaterpress) / (Major principleσ)	0	<0.1	0.1-0.2	0.2-0.5	>0.5
		General conditions	Completely dry	Damp	Wet	Dripping	Flowing
	Rating		15	10	7	4	

Table 3: Rock mass classes (after Bieniawski, 1989)

Rating	100-81	80-61	60-41	40-21	<20
Class No	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

Slope Mass Rating (SMR)

The SMR is a quantifying method applied on rock masses or rock slope to evaluate the stability conditions of the rock slope. First developed by Romana (1985), it is an empirical equation, modified after the Rock Mass Rating proposed by Bieniawski (1989), by adding factorial adjustment factors for the discontinuity orientation. The slope mass rating is the most comprehensive and widely used technique for rock slope assessment (Umrao et al., 2011).

SMR is obtained from RMR by adding a factorial adjustment factor depending on the joint-slope relationship and adding a factor for the natural slope. Adjustment ratings F1, F2, and F3 for joints are evaluated depending on the joint direction (αj), slope direction (αs), joint angle (βj), and slope angle (βs) (Romana, 1985). The value of F4 is taken corresponding to natural slopes. Here,

- SMR = RMR + (F1 × F2 × F3) + F4F1 depends on the parallelism between strikes of joints and slope faces. Values range from 1.00 to 0.15.
- F2 refers to joint dip angle in the planar mode of failure. Its value ranges from 1.00 to 0.15.
- F3 reflects the relationship between slope face and joint dips.

- F4 denotes the adjustment factor for the method of excavation that has been fixed empirically. The adjustment rating and stability classes are represented in Tables 4 and 5 respectively.

Table 4: SMR rating system (after Romana, 1985)

Case	**Very favourable**	**Favourable**	**Fair**	**Unfavourable**	**Very unfavourable**
P α_i- α_s T α_i-α_s- 180° P/T	>30° 0.15	30°-20° 0.40	20°-10° 0.70	10°-5° 0.85	<5° 1.00
P β_j-β_s P F2= T F2	<20° 0.15 1	20°-30° 0.40 1	30°-35° 0.70 1	35°-45° 0.85 1	45° 1.00 1
P β_j-β_s P β_i- β_s T F3	>10° <110° 0	10°-0° >110°-120° -6	0 >120° -25	0°-(-10°) -50	<-10° -60
F4	Natural slope +15	Pre- splitting +10	Smooth blasting +8	Regular blasting 0	Deficient blasting -8
P = planar failure		α_s = slope direction		α_j = joint dip direction	
T = toppling failure		β_s = slope dip		β_j = joint dip	
SMR = RMR + (F1 × F2 × F3) + F4					

Table 5: SMR classes (after Romana, 1985)

Class No	**V**	**IV**	**III**	**II**	**I**
SMR	0-20	21-40	41-60	61-80	81-100
Description	Very poor	Poor	Fair	Good	Very good
Stability	Very unstable	Unstable	Partiallystable	Stable	Fully stable
Failures	Large planar or soil-like	Planar or large wedge	Some joints or many wedges	Some blocks	None
Support	Re-excavation	Extensivecorrective	Systematic	Occasional	None

Kinematic Analysis

Kinematic analysis in landslide studies is a method for analysing potential modes of rock slope failure that may occur due to unfavourable orientation of structural discontinuities (joints, faults, foliations, beddings). The resulting failures may be a planar failure, wedge failure and toppling failure. A planar failure occurs when the failure plane strike parallel or nearly parallel (±20°) to the strike of slope and when dip of failure plane is less than the inclination of slope and greater than the angle of friction along the failure plane. For a wedge failure to occur, the difference between plunge direction of the line of intersection of two discontinuity planes and the direction of inclination of slope face should be less than 20° and the plunge amount of line of intersection of two discontinuity planes should be less than the inclination of slope face but more than the friction angle of slope material. The prerequisite condition for a toppling failure to occur is that, the strike of dominant discontinuity, basal plane of separation and that of the slope face should be parallel or if skewed then at most by 10° to each other (Anbalagan et al., 2007). Graphically, kinematic analysis can be carried out by plotting the discontinuity planes on a circular graph called stereonet or stereogram, based on their orientations in terms of dip direction and inclination of dip. The orientations of the discontinuities are represented on a stereonet in the form of great circles, poles, or dip vectors. Clusters of poles of discontinuity orientations on stereonets are identified by visual investigation or using density contours on stereonets. The direction of a slope failure can be deduced from the stereonet.

The present study utilises RMR, SMR and kinematic analysis to determine the geomechanical influence in the Noklak landslide. Altogether six locations (L1 to L6) were studied. The bedding trends and joints were recorded to identify the possible mode of failure. Fifty rock samples were collected from four locations for determination of the compressive rock strength using the PLT, while the strength of intact rocks in two locations (Table 6) are determined in the field itself using Schmidt hammer (Fig. 2, Fig. 3, Fig. 4). The bedding and joint trends were plotted in Rocscience Dips software (Rocscience Inc., 1998) to construct contour diagram, stereogram and rose diagrams (Fig. 5).

Table 6: Sampling location

Sample Point (L)	Location	Joints measured	Schmidt hammer testreadings	Samples collected
L1	26° 12' 6.75"N 95° 1' 3.60"E	212	--	12
L2	26° 12' 3.48"N 95° 1' 4.61"E	161	--	14
L3	26° 11' 58.74"N 95° 01' 01.64"E	223	--	13
L4	26° 11' 55.28" N 95° 0' 58.90" E	193	--	11
L5	26° 11' 56.67"N 95° 0' 56.14"E	237	10	--
L6	26° 11' 28.69" N 95° 0' 7.38" E	169	19	--

Fig. 2: Schmidt Hammer

Fig. 3: Fractures and joints

Fig. 4: PLT

RESULTS

RMR & SMR

L1

Point load index (PLI) = 0.47MPa, since PLI value is less than 1MPa (Table 1)

Hence using the UCS equation,UCS = 14.4 × 0.47 = 6.76 MPa

RQD = 115- 3.3 × 55 = -66.5

αj (joint direction) = 310, αs (slope direction) = 243

βj (joint angle) = 55, βs (slope angle) = 35

Values are plotted in Table 2.7a

L2

PLI= 0.53 MPa. since PLI value is less than 1MPa (Table 2.1)

UCS = 14.4 × 0.53 = 7.632 MPa, RQD = 115- 3.3 × 60.66 = -85.178

αj (joint direction) = 283, αj s (slope direction) = 262

βj (joint angle) = 80, βs (slope angle) = 36
Values are plotted in Table 2.7a

L3

PLI = 0.51 MPa. since PLI value is less than 1MPa (table 1)
UCS = 14.4 × 0.51 = 7.344 MPa, RQD = 115- 3.3 × 36 =3.8
αj (joint direction) = 238, αs (slope direction) = 252
βj (joint angle) = 86, βs (slope angle) = 31
Values are plotted in Table 7a

Table 7a: SMR System (after Romana 1985)

	L1		L2		L3	
	Value or Condition	Rating	Value or Condition	Rating	Value or Condition	Rating
1. UCS	6.76 MPa	2	7.63 MPa	2	7.34 MPa	2
2. RQD	-66.5%	3	-85.178%	3	-3.8%	3
3. Spacing of joints	35.69 mm	5	46 mm	5	130 mm	8
4.Condition of joints	Slightly rough surface Separation <1 mm; Highly weathered walls	20	Slightly rough surface Separation <1 mm; Highly weathered walls	20	Slickenside surface; continuous joints; separation <5 mm	10
5.Groundwater condition	Damp	10	Damp	10	Damp	10
RMR	= (1+2+3+4+5)	***40***	= (1+2+3+4+5)	***40***	= (1+2+3+4+5)	***33***
6. $F_1 = \alpha j - \alpha_S$)	71°	0.15	19°	0.7	-15°	1
7. $F_2 = \beta j - \beta j$	54°	1	80°	1	86°	1
8. $F_3 = \beta j - \beta_S$ for plane failure where β_S= dip/angle of slope	19°	0	43°	0	51°	0
9. F_4 = Adjustment factor	Pre-splitting	10	Pre-splitting	10	Pre-splitting	10
SMR = *RMR+(F1xF2xF3)+F4*	40 + {0.15 × 1 × 0} + 10	50	40 + {0.7 × 1 × 0} + 10	50	33 + {1 × 1 × 0} + 10	43
10. Class	**III**		**III**		**III**	

L4

PLI= 0.52 MPa. since PLI value is less than 1 MPa (Table 1)

UCS = 14.4 × 0.52 = 7.49 MPa, RQD = 115- 3.3 × 23.5 = 37.45

αj (joint direction) = 74, αs (slope direction) = 260

βj (joint angle) = 77, βs (slope angle) = 34

Values are plotted in Table 7b

L5

Rebound number, R_n = 19.9

Since the lithology in the study area is siltstone, density ρ = 2.6

UCS = $6.9 \times 10^{(0.16 + 0.0087 \times 19.9 \times 2.6)}$

= 28.10 MPa

RQD = 115 - 3.3 × 33 = 6.1

αj (joint direction) = 255, αs (slope direction) = 262

βj (joint angle) = 80, βs (slope angle) = 37

Values are plotted in Table 7b

L6

Rebound number, R_n = 28.84

Since the lithology in the study area is shale, density ρ = 2.15

UCS = $6.9 \times 10^{(0.16 + 0.0087 \times 28.84 \times 2.15)}$

= 33.79 MPa

RQD = 115 - 3.3 × 38 =-10.4

αj (joint direction) = 175, αs (slope direction) = 120

βj (joint angle) = 85, βs (slope angle) = 75

Values are plotted in Table7b

Table 7b: SMR System (after Romana 1985)

	L4		L5		L6	
	Value or Condition	Rating	Value or Condition	Rating	Value or Condition	Rating
1. UCS	7.49 MPa	2	28.10 MPa	2	33.79 MPa	4
2. RQD	37.45%	8	6.1%	3	-10.4%	3
3. Spacing of joints	92.37mm	8	225mm	10	60mm	8
4. Condition of joints	Soft gouge <5 mm or Separation >5 mm Continuous joints	0	Slickenside surface; <5 mm thick separation 1-5 mm; continuous joints	10	Slightly rough surface Separation <1 mm; Highly weathered walls	20

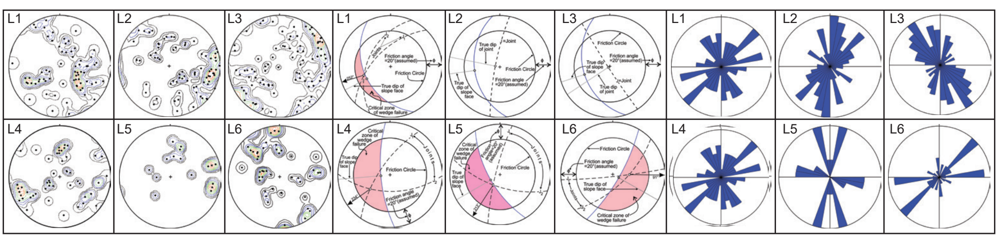
5.Groundwater condition	Completely dry	15	Damp	10	Flowing	0
RMR	= (1+2+3+4+5)	**33**	= (1+2+3+4+5)	**35**	= (1+2+3+4+5)	**35**
6. $F_1 = \alpha_j - \alpha_s$	-192°	1	44°	0.15	54°	0.15
7. $F_2 = \beta_j - \beta_j$	77°	1	80°	1	85°	1
8. $F_3 = \beta_j - \beta_s$ for plane failure where β_s = dip/angle of slope	7°	-6	15°	0	10°	-6
9. F_4 = Adjustment factor	Pre-splitting	10	Pre-splitting	10	Pre-splitting	10
***SMR** = RMR+(F1xF2xF3)+F4*	*33 + {1 × 1 × (-6)} + 10*	37	*35 + {0.15 × 1 × 0} + 10*	45	*35 + {0.15 × 1 × -6} + 10*	44.10
10. Class	**IV**		**III**		**III**	

Table 8: SMR class and description of different locations

Sampling Points	Class	Description
L1	III	Fair rock; partially stable slope prone to failure by some joints or many wedges; requires systematic measures
L2	III	Fairrock; partially stable slope prone to failure by some joints or many wedges; requires systematic measures
L3	III	Fairrock; partially stable slope prone to failure by some joints or many wedges; requires systematic measures
L4	IV	Poor rock; unstable slope prone to both planar and wedge failure; requires extensive corrective measures.
L5	III	Fair rock; partially stable slope prone to failure by some joints or many wedges; requires systematic measures
L6	III	Fair rock; partially stable slope prone to failure by some joints or many wedges; requires systematic measures

Kinematic Analysis

Fig. 5: Contour diagrams, Stereographic projection and Rosette generated from rock joints



Results from Stereographic projection and Rose diagram generated from the joints of the study area:

L1

Dominant trends are NE-SW (regional fault and thrust direction) and NNW-SSE antithetic shears (possibility of strike-slip faults). Minor WNW-ESE trend seen is due to synthetic shears. Here, the dominant joint sets J_1 (54° due 314°) and J_2 (41° due 294°) with respect to slope face (35° due 243°) intersect to produce wedge in the direction 252° (Fig. 2.5). The plunge of intersection lines of discontinuity lies within the shaded region and is less than the dip angle of the slope face. This indicates wedge mode of failure according to Hoek and Bray (1981).

L2

The dominant trend is NNE-SSW (most likely regional fault and thrust direction).In slope L2, the principle joint set J (80° due 281°) strike (±19°) with respect to slope face (37° due 262°) (Fig. 2.5).

L3

The dominant trend is NW-SE. The joints in this direction are likely to be coupled with normal fault. In L3, the primary joint set J (86°/237°) strike (±15°) with respect to slope face (35° due 252°) (Fig 5). Both slope L2 and L3 indicate planar failure.

L4

The trend is approximately NNW-SSE. The area may be affected by some joints. However, any displacement along this direction could develop sinistral strike-slip faults. The dominant joint sets J_1 (55° due 181°) and J_2 (67° due 304°) with respect to slope face (34° due 260°) intersect to produce wedge in the direction 234° (Fig. 5). The plunge of intersection lines of discontinuity lies within the shaded region and is less than the dip angle of the slope face indicating wedge mode of failure.

L5

The trend is approximately NNW-SSE. The area may be affected by some joints. However, any displacement along this direction could develop sinistral strike-slip faults. In this location, joint sets J_1 (80° due 254°) and J_2 (80° due 284°) with respect to slope face (37° due 262°) intersects to produce wedge in the direction 213° (Fig. 5). The plunge of intersection lines of discontinuity lies within the shaded region and is less than the dip angle of the slope face indicating wedge mode of failure.

L6

The trends are approximately NE-SW and E-W. Here, joint sets J_1 (85° due 174°) and J_2 (64° due 137°) with respect to slope face (75° due 120°) intersect to produce wedge in the direction 91° (Fig. 5). The plunge of intersection lines of discontinuity lies within the shaded region and is less than the dip angle of the slope face indicating wedge mode of failure.

Table 9: Data description showing the relation between joints and slope

Location	**L1**	**L2**	**L3**	**L4**	**L5**	**L6**
Slope Orientation	35°/243°	37°/262°	35°/252°	34°/260°	37°/262°	75°/120°
Orientation of Principle Joint Sets	J_1=54°/314° J_2=41°/294°	J=80°/281°	J=86°/237°	J_1=55°/181° J_2=67°/304°	J_1=80°/254° J_2=80°/284°	J_1=85°/174° J_2=64°/137°
Failure mode	Wedge	Planar	Planar	Wedge	Wedge	Wedge
Data format	Dip/dip direction					
Magnetic declination = (-ve) 0.433° (west declination of the study area)						

Discussions

Out of the six locations studied, RMR and SMR results of five locations classify the rocks to be normal rock, falling under class III, which is indicative of a partially stable slope that can be prone to failure due to joints (Karaman et al., 2013), while one location is classified under class IV indicating weak rocks. Gravity is the driving force of landslides, but its effectiveness in producing landslides depends on certain other factors. The slope in the study area has very few exposures of well-bedded rocks and is composed mostly of soil and rock debris and hence from a lithological point of view, the slope may be assumed to be fragile and prone to failure. The shales, which are the dominant rock type in the study area, are highly sheared, pulverised and weathered, making them very weak and vulnerable to erosion(Varnes et al., 1978). Structurally, the rocks in the area are dissected by number of joint sets, minor folds and faults. Analysis of rose diagrams and the stereographic projections of the area show at least four dominant joint sets which makes the rocks prone to a planar or wedge type of failure. After a thorough and detailed investigation of the lithology and based on the geo-mechanical parameters, it can be concluded that the slopes in the study area are fairly stable which however are susceptible to failure in the presence of discontinuities such as joints, their distribution and interaction. The slope instability may also have been aggravated due to the presence of weak slope materials and absence of firm rock bedding (Hudson et al., 1997; Hoek et al., 1998). Furthermore, poor drainage and the introduction of water by anthropogenic activities and heavy precipitation may be a triggering factor (Zezere et al., 1998; Wu, 2003). One important factor that might be controlling the slope instability in the area may be attributed to the neo-tectonic activities (Deng et al., 2000; Korup et al., 2007). Study of satellite image shows the presence of a major lineament trending NE-SW, cutting across the Noklak town which trends along the general thrust direction of the region. The Kiamong river has carved its channel along this fault. Along the stream channel, the continual occurrence of slicken sidelends evidence to faulting in the area. The region is traversed by a number of other lineaments. Four lineaments trending parallel to each other, are oriented NW-SE, which is the normal fault setting in the region. The township is dissected by two of these faults. On the northern and southern extremities of the study area, two other prominent lineaments are seen trending parallel to each other along ENE-WSW(Fig. 6), which appear to be hybrid fractures resulting from the complex interplay of stresses.

Fig. 6: Lineament map (Thong, 2019)`

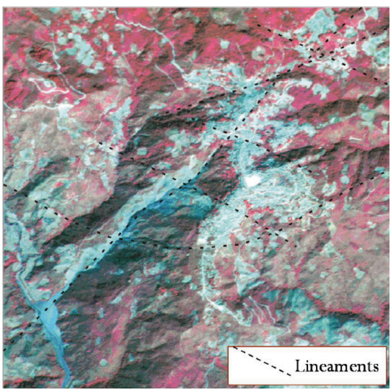


Fig. 7: View of the slide affected area

Mitigation Measures

Noklak town is situated on a tectonically unstable hilly region and developmental activities associated with urbanization imposes great stress on the slopes leading to reduction of the shear strength and resulting in landslides. It is, therefore, necessary to have a proper town planning, develop master plans especially for designing appropriate drainage systems including stormwater drains to ensure the minimal flow of water to the slide affected area. Unscientific and rampant developmental activities including construction of heavy RCC buildings, road cuttings, clearing of vegetative covers for farming etc. in the vicinity of the slide should be restricted. Plantation of well spread, deep-rooted vegetation such as the vetiver grass may impart shear strength to the slope by holding the slope material together. Construction of check dams and embankments should be taken up to reduce toe erosion along the Kiamong river. To reduce the impact on life and properties from the threat of this landslide, public awareness should be generated as well as developing an early warning system.

Conclusion

The present study considered the role of rock mechanics to study the complex landslide at Noklak town. The results show poor rock in only one location and fair rock with the partially stable slope in the other five locations. However, the result also indicates that the presence and distribution of joints would make the slope vulnerable to planar or wedge type of failure. This study concludes that the occurrence of a large amount of rock joints in differing orientations has played a vital role as a causative factor in some cycle of slope movement in this decades-old Noklak landslide. Further investigation to ascertain the role of neotectonics, soil mechanics, etc. is required.

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