STABILITY ANALYSES OF TWO FRESH CUT SLOPES ALONG NH 2, MERIEMA, NAGALAND

Supongtemjen[1], Temsulemba Walling[2], Sohi Tep[2], and Glenn T. Thong[2]*

[1]GIS & RS Center, Planning & Coordination Department, Govt. of Nagaland, Kohima

[2]Department of Geology, Nagaland University, Kohima Campus, Meriema

*Corresponding Author: glen2t03@yahoo.com

ABSTRACT

The National Highway 2 is being widened, and in the process steep slopes have been left without any support, which has led to sliding of debris. Hence, studies were carried out at two selected sites to determine the causes and modes of failure with a view to provide appropriate mitigation measures. This highway has been cut through the Disang Group of rocks, which comprise thick beds of shale intercalated with thin flaggy sandstone. The rocks are sheared and crushed to varying degrees in response to ongoing tectonism. The upper horizons are made up of partially weathered shale capped by debris and loose soils. Two to three prominent sets of joints are noted.

Consistency of soils analysed from samples collected from the slide zones indicate low liquid limit and high plasticity. About forty rock samples were collected from the periphery of each slide zone for analyses. Point load tests indicate weak strengths of 1.53 and 1.88 MPa. Rock and slope mass ratings indicate more or less weak rocks and unstable slope conditions, respectively. About 200 joint attitudes were measured from each site to identify the dominant joint sets. These plotted in stereographic projections against slope indicate possible wedge failure. A rose diagram of the joints suggests complex deformation, including shearing of the rocks.

Keywords: Stability analyses, Mitigation measures, NH 2, Meriema, Nagaland

INTRODUCTION

Landslides and other forms of surface instability have posed major challenges to the country in recent years. Slope failure is common in most mountainous terrains and affects life and economy of both urban and rural areas, besides disturbing the ecosystem. Topography, lithology, geologic structures, and groundwater are important natural initiators of landslides. Haphazard and unscientific developmental activities worsen existing stability conditions and often, in combination with heavy rainfall trigger landslides (Chen and Lee, 2002). Gray (1973), Swanson and Dyrness (1975), Swanston and Swanson (1977), and Valdiya (1987) blame unscientific road construction for initiating landslides. For sustainable development in hilly terrain, study of cut-slopes is therefore important. To ensure stability of slopes, particularly along highways, excavations require an evaluation of the structures affecting rocks. Rock and soil cuts along highways should be made with appropriate safety designs in place to avoid slope failure.

Nilsen et al. (1976) are of the opinion that a combination of various factors such as soils and surface deposits, types and properties of underlying bedrock, angle and direction of slope, amount of rainfall, placement of cuts, ancient landslide deposits, etc. are responsible for landslides. Lithology and structure play a vital role in the development and disposition of slopes and instability pattern in any area (Emelyanova, 1977). Veder and Hilbert (1980) opine that a landslide will develop at the toe of a slope as soon as the driving forces exceed the resisting forces. Slope failure takes place when the critical slope angle is exceeded. This angle depends on the frictional properties of slope material. It is estimated that most landslide events have occurred on slopes greater than 30° (Terzaghi, 1950). Shearing stresses build up with increasing inclination and height of sloping surfaces. Failure will occur when shearing stresses exceed the shearing strength of the slope forming material. The amount of shearing and fracturing and the attitude of beds or joints with relation to slope geometry are important criteria in determining slope stability conditions. Slope instability is commonly influenced by huge accumulations of debris in the head regions of slides and increase in moisture content (Towhata, 2007).

Climate plays a vital role in slopes stability. Landslides occur frequently due to climatologic and geologic conditions with high tectonic activities. Slopes excavated during the dry season are usually stable but gradually weaken during the monsoon. Incessant rainfall often acts as a trigger for slope failure (Aier et al., 2012). Water enters pores and cracks of slope material and causes swelling, which ultimately leads to decrease of shearing strength (Nishida et al., 1979; Crozier, 1989). Such slopes may suddenly become unstable. Dortch et al. (2008) state that heavily fractured bedrock and varying lithologies can enable an enhanced monsoon rainstorm to trigger large landslides. Veder and Hilbert (1980) insist on the importance of clays in landslide studies, because their cohesiveness and shear strength fall in the presence of water.

Nagaland is a mountainous region that is affected by landslides every monsoon. The Naga Hills, trending approximately NE-SW, is part of the northern extension of the Indo-Myanmar Range (IMR). This region is part of Cretaceous-Tertiary orogenic upheavals of a major mobile belt of the earth (Mathur and Evans, 1964; Directorate of Geology and Mining, 1978; Ghose et al., 2010). Due to subduction of the Indian Plate beneath the Burma Plate, rocks in the region have been uplifted (Imchen et al., 2014). This tectonically complicated region represents a relatively young immature mountainous terrain. The subduction and uplift processes that began during the Cretaceous is believed to be continuing (Nandy, 1976; Verma, 1985; Bhattacharjee, 1991; Aier et al., 2011). This has caused extensive shearing, fracturing, and jointing of the rocks. Petley and Reid (1999) opine that landslides are inevitable where mountain chains are being uplifted. Various geomorphic processes have further weathered and eroded the weakened rocks of the region leading to large scale slope instability (Aier et al., 2009).

The former State Highway No. 1 was upgraded to National Highway (NH) and re-designated NH 2. Consequently, widening of this narrow highway has begun to that of four-lane category. However, slope excavations have been taken up very indiscriminately without any engineering considerations. As such, some sections of this highway are vulnerable to various forms of mass wasting. This highway runs through thick piles of Disang shale intercalated with thin beds of flaggy sandstone and siltstone. These rocks are sheared and crumpled to varying degrees. The weakened shales are partially weathered to clays. Such weak rocks, besides adverse hydrogeological conditions and heavy and prolonged rainfall are prone to instability. Excavations for widening the highway are being undertaken without consideration for mitigation measures resulting in some slopes being left as high as 10-18 m and at angles of 70º-80º without any support. It is thus, for such reasons that portions of these roads are weak or affected by debris slides. Due to the importance of this highway and high risk in parts of the study area, studies have been carried out to determine the causes of failure in the affected areas and probable modes of failure in the zones that are still unaffected, with a view to recommend appropriate mitigation and/or preventive measures (Supongtemjen, 2013).

The study area (Fig. 1) is part of the Survey of India topographic map no. 83 K/2 NW. In the study area fresh cut slopes do not have adequate roadside drainage. Moreover, continued debris slides litter the road leading to rapid deterioration of the bitumen. So roadside drains and favorably modified slopes or slopes with adequate support that prevent debris from straying down into the drains and onto the highway are important.

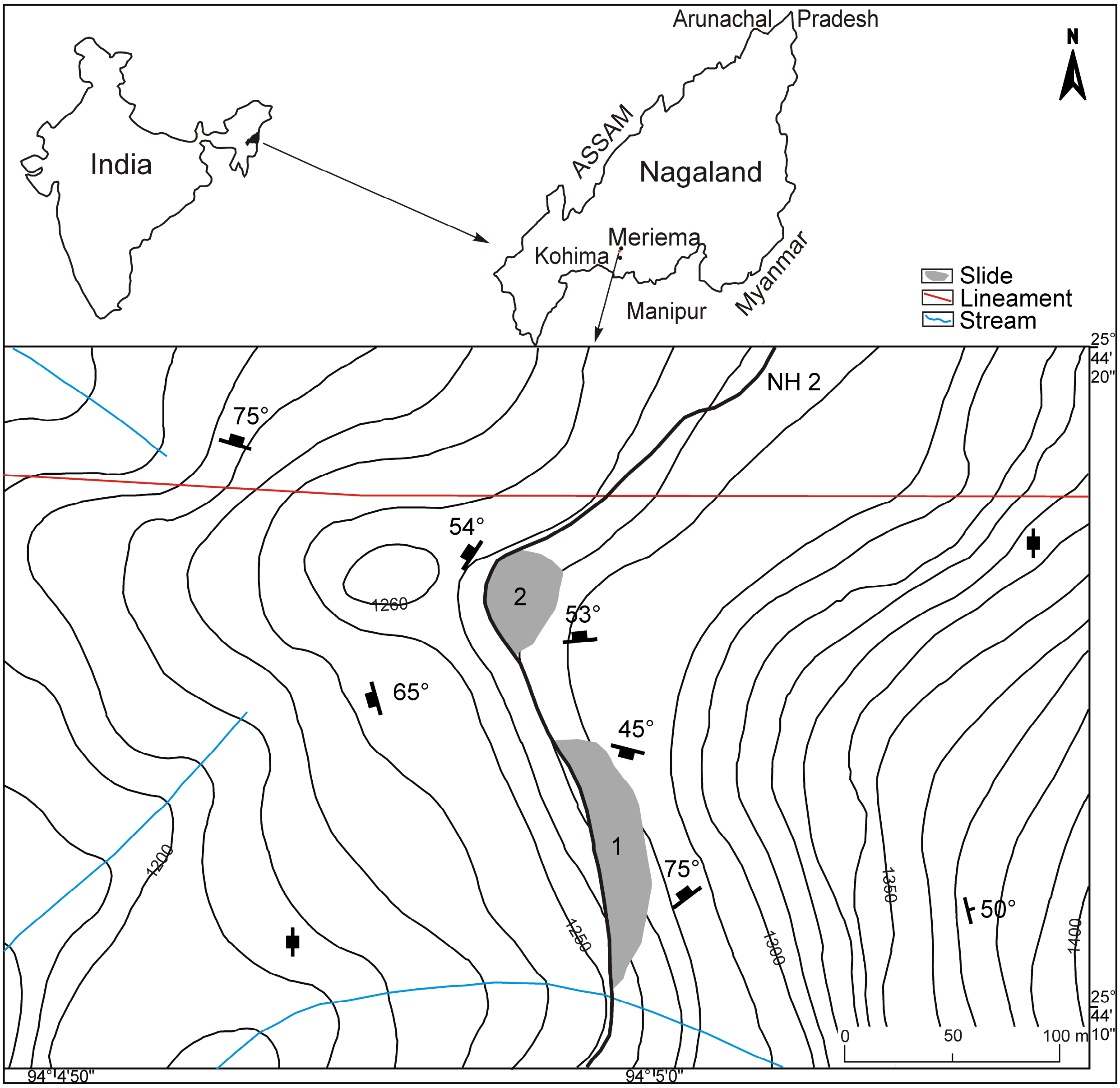

Fig. 1. Location map

METHODOLOGY

Geotechnical analyses

Soils of the two slide zones were analysed to evaluate the geotechnical parameters. Soil samples were collected and their moisture contents determined. This was followed by determination of the liquid and plastic limits, and shrinkage factors by standard techniques. The index properties were then calculated.

RMR / SMR

Studies of fresh-cut slopes along this highway enable an understanding of stability conditions in the area. The present study is based on the slope mass rating (SMR) of Romana (1985), which is a product of the rock mass rating (RMR) of Bieniawski (1979). The RMR was computed by adding rating values for five parameters including strength of intact rock, rock quality designation (RQD), spacing and condition of discontinuities, and water inflow through discontinuities. The RQD index developed by Deere et al. (1967) provides a quantitative estimate of rock mass quality. Palmström (1982) estimated RQD from the number of joints per unit volume. The condition of joints was inferred from the inherent surface smoothness or unevenness and waviness relative to the plane of the joint. Joint roughness can be estimated numerically from the joint roughness coefficient (JRC). The JRC was estimated by comparing the appearance of a discontinuity surface with the standard profile of Barton and Choubey (1977). Groundwater condition is an important parameter in the assessment of stability conditions of a slope. Groundwater condition was visually estimated to provide ratings. The algebraic sum of these five parameters gives the RMR values for a particular slope facet. For slope stability assessment, stereographic analysis of discontinuities was carried out for determining the mode of failure and adjustment ratings. The adjustment ratings F_1, F_2, and F_3 for joints were evaluated depending on joint trend (α_j), slope face trend (α_s), joint dip angle (β_j), and slope angle (β_s) after Romana (1985). The value of F_4 was taken corresponding to natural slopes.

The SMR was obtained from the RMR by adding a factorial adjustment factor depending on the joint-slope relationship and adding a factor for the natural slope. Here, SMR = RMR+ ($F_1 x F_2 x F_3$) +F_4

1) F_1 depends on parallelism between strikes of joints and slope faces. Values range from 1.00 to 0.15. These values match the relationship $F_1=(1\text{-SinA})^2$, where A denotes the angle between the strikes of slope faces and joints.

2) F_2 refers to joint dip angle in the planar mode of failure. Its value ranges from 1.00 to 0.15 and matches the relationship $F_2=tg^2Bj$, where Bj denotes the joint dip angle. For the toppling mode of failure F_2 remains 1.00.
3) F_3 reflects the relationship between slope face and joint dips.
4) F_4 denotes the adjustment factor for the method of excavation that has been fixed empirically.

Kinematic analyses

Kinematic analyses have been performed for determination of possible failure mode along fresh-cut slopes of the highway. Angular relationships between discontinuities and slope surfaces were applied to determine the potential and modes of failures after Kliche (1999). Joint trends were taken from each location and plotted in GEOrient software (R.J. Holcombe, University of Queensland, Australia) to construct polar density and contour diagrams to determine the dominant joint sets that control instability. These were used to generate stereographic projections to ascertain type of failure mode. Data generated from joint projections were used for kinematic analyses using the Markland test (Markland, 1972), which was modified by Hocking (1976), Cruden (1978), and Hoek and Bray (1981). Using this method the probable mode of failure at each location was determined.

LOCATION 1

Introduction

This area is located at 25°44'12.41" N latitude and 94°04'59.52" E longitude (Fig. 1 - 1). Debris falls and slides are fairly common with about 110 m of the road being affected in 2007 (Fig. 2). The area is made up of partially weathered shale capped by weak soil, the thickness of which ranges from 2 to 3 m. The rocks are jointed, fragmented, and weathered due to which debris continuously slides downhill to block the roadside drain and part of the highway. The bedrocks dip about 30° towards 135° SE. Three prominent sets of joints are noted. The first set of joints trends NW-SE dipping at angles of 45° towards SW. The second set trends N-S, dipping towards 270° W and the third set trends NNE-SSW, dipping 72° NNW. The triggering factor for instability in this section of the highway is a combination of steep slope, adverse lithology, and structure. The height of the unstable slope here is about 10 m with an inclination of 75º. Such slopes made up of partially weathered shale overlain by loose debris and soil are highly prone to slope failure.

Fig. 2. Weak slope material

Geotechnical analyses

The liquid limit value (Table 1) indicates that the soils are weak and unstable. The plasticity index suggests that the soils are plastic. The liquidity and consistency indices indicate a semisolid state. Toughness index values between 0 and 3, according to Murthy (2001), indicate clayey soils. The soils may be a natural admixture of inorganic clays and silts of medium plasticity. Any disturbance with increase in the natural moisture content can therefore cause the soil to flow. Hence, this area is unstable, especially during the monsoon when the amount of rainfall is high.

Table 1. Consistency limits

Moisture content %	Liquid limit %	Plastic limit %	Plasticity index %	Liquidity index %	Flow index %	Toughness index %	Shrinkage limit %	Shrinkage ratio %	Volumetric shrinkage %	Consistency index %
25.78	44.50	26.89	17.61	-0.06	16.50	1.07	22.98	1.58	59.80	1.06

SMR and KA

Forty five rock samples were collected from the site for determination of their strengths. The RMR value of 39 (Table 2) obtained indicates weak rock. SMR values fall in Class IV indicating unstable slope conditions. This also indicates potential planar or wedge failure.

Table 2. Slope mass rating

	Value or Condition	Rating
1. Point Load Test	1.53 MPa	4

2. RQD	32.5%	8
3. Spacing of joints	40 mm	5
4. Condition of joints	Slightly rough; separation <1 mm; soft joint wall rock	12
5. Groundwater condition	Dry	10
RMR	***= (1+2+3+4+5)***	***39***
6. $F_1 = (1 - \mathrm{Sin}I\alpha_j - \alpha_s I)^2$	30°	0.40
7. $F_2 = \mathrm{Tan}^2\beta_j$ or $F_2 = 1$ for toppling	35°	0.70
8. $F_3 = I\beta_j - \beta_s I$ for plane failure $= I\beta_j + \beta_s I$ for toppling where β_s= dip/angle of slope	-30°	-60
9. F_4 = Adjustment factor	Pre-splitting	10
***SMR** = RMR+(F_1xF_2xF_3)+F_4*	*39+{0.40x0.70x(-60)}+10*	***32.2***
10. Class	IV	

200 joint attitudes taken in the field were used to construct pole density (Fig. 3a) and contour diagrams (Fig. 3b) and from which two dominant joint sets that lend instability to the slope were identified. These were plotted in a stereographic projection against slope (Fig. 3c).

1. Slope : 65°→230°
2. J_1 : 45°→210°
3. J_2 : 75°→292°

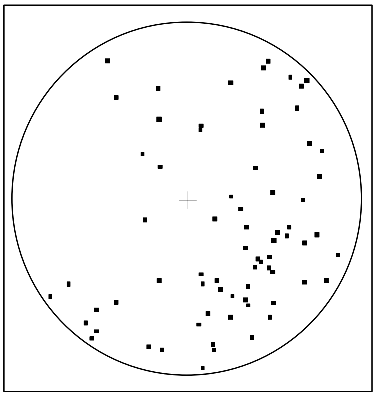

Fig. 3a. Pole diagram

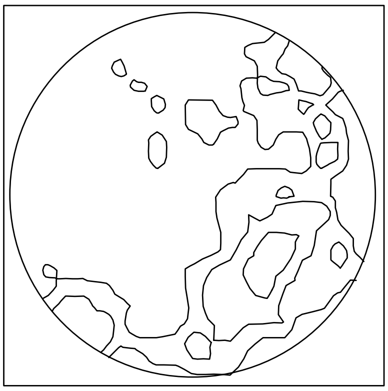

Fig. 3b. Contour diagram

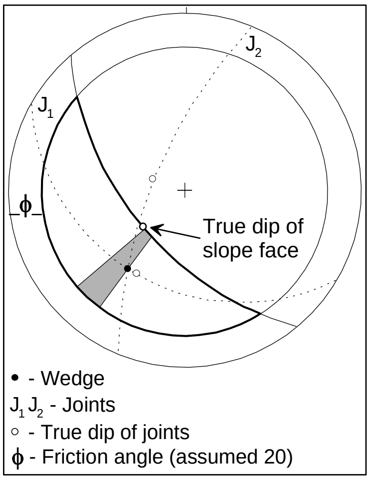

Fig. 3c. Stereogram

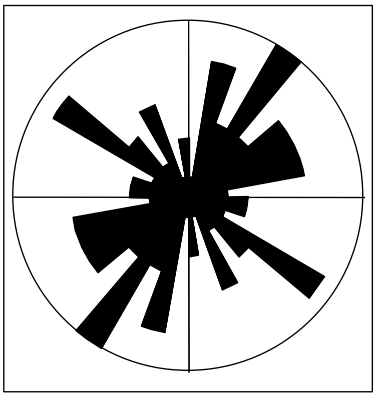

Fig. 3d. Rose diagram

The figure shows the development of a distinct wedge due to the intersection of these two joint sets. Both the true dips of these joints lie outside the shaded area. The intersection of J_1 and J_2 forms a double plane wedge. A rose diagram (Fig. 3d) indicates thrusts (NE-SW) and possible tensile fractures (NW-SE) affecting the rocks.

Recommendations

a. Reduction of slope angle in the upper soil horizon and removal of 3 m of the top soil is recommended (Fig. 4).
b. A geotextile cover and grass plantation on the excavated soil will greatly help.
c. About 100 m length of the rock section treated with GI wire netting will provide stability to the slope.
d. Proper roadside drainage is necessary.

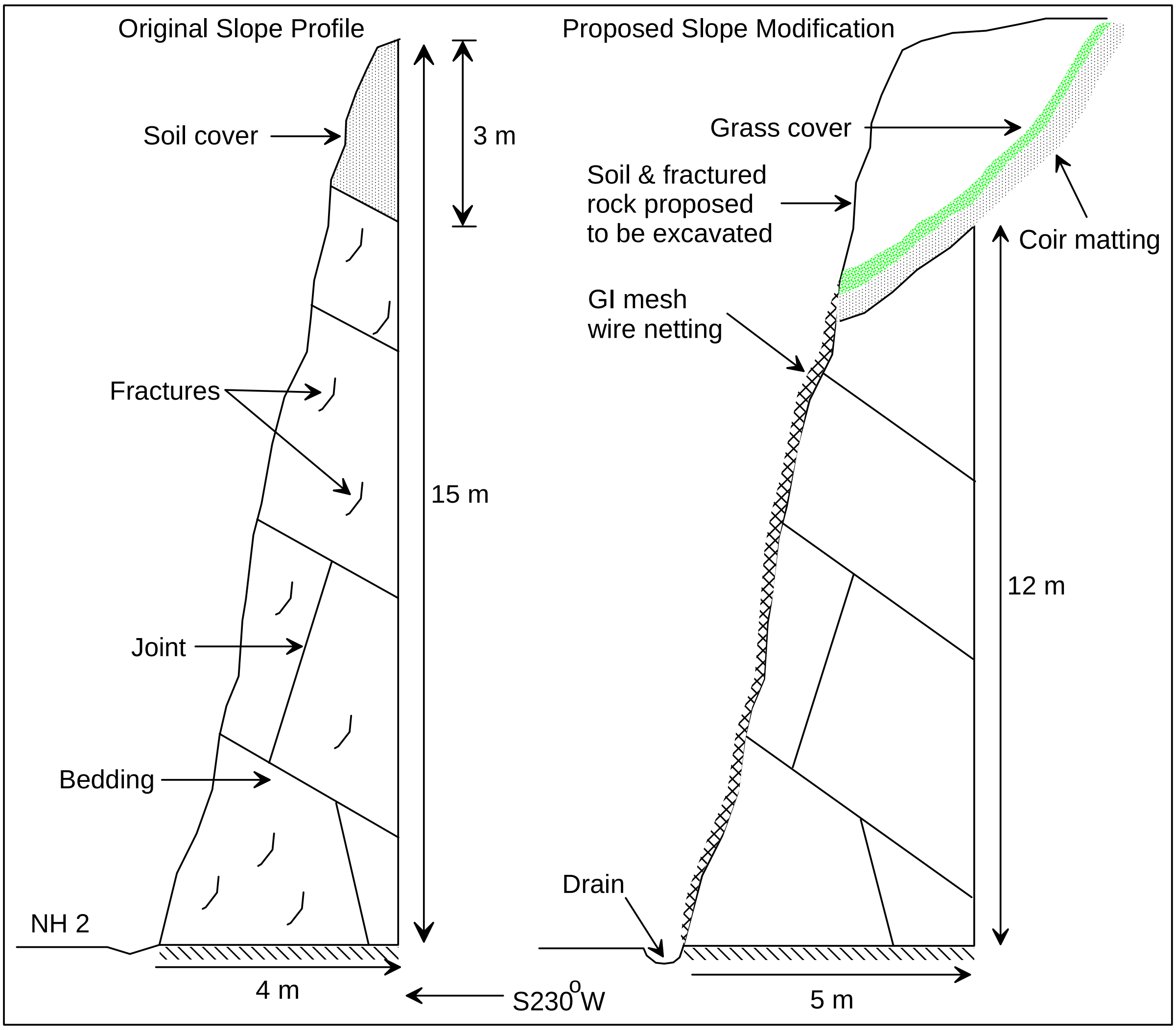

Fig. 4. Proposed slope modification / mitigation measures

LOCATION 2

Introduction

This area located at 25°44'18.08" N latitudes and 94°05'1.05" E longitudes (Fig. 1 - 2) was affected by minor but continuous debris slides (Fig. 5). The slope materials making up the area are crumpled Disang shale. The upper horizons comprise debris and loose soil. Bedrocks are not exposed along the section but outcrops in the vicinity show a number of joints. Two prominent sets of joints control rock behavior. The first set of joints trends ENE-WSW, dipping at an angle of 54° towards NNW. The second set trends N-S and dips 53° towards the west. The cause of this instability is due to weak lithology on a high (10 m) and steep slope with a gradient of 70º.

Fig. 5. Weak hill slope

Geotechnical analyses

The liquid limit (Table 3) indicates that the soils are weak and unstable. The plasticity index indicates that the soils are in a highly plastic state. Such soils are vulnerable to plastic deformation, which can cause slope failure as mudflows. The liquidity and consistency indices indicate soft and plastic soils. The toughness index indicates clayey soils. Hence, when the moisture content, such as during the monsoon increases, any disturbance, including steep slopes, can cause the soil to flow.

Table 3. Consistency limits

Moisture content %	Liquid limit %	Plastic limit %	Plasticity index %	Liquidity index %	Flow index %	Toughness index %	Shrinkage limit %	Shrinkage ratio %	Volumetric shrinkage %	Consistency index %
29.80	45.50	26.34	19.16	0.18	13	1.47	18.26	1.64	70.95	0.82

SMR and KA

Forty rock samples were collected from the site for strength determination. The RMR value of 46 indicates fair weak rock. SMR values fall in Class III, which indicates partially stable conditions (Table 4).

Table 4. Slope mass rating

	Value or Condition	Rating
1. Point Load Test	1.88 MPa	4
2. RQD	49%	10
3. Spacing of joints	50 mm	10
4. Condition of joints	Slightly rough; separation <1 mm;	12

	soft joint wall rock	
5. Groundwater condition	Dry	10
RMR	*= (1+2+3+4+5)*	***46***
6. $F_1 = (1\text{-Sin}I\alpha_j - \alpha_s I)^2$	0.85	0.85
7. $F_2 = \text{Tan}^2\beta_j$ or $F_2 = 1$ for toppling	0.70	0.70
8. $F_3 = I\beta_j - \beta_s I$ for plane failure $= I\beta_j + \beta_s I$ for toppling where β_s= dip/angle of slope	4	-6
9. F_4 = Adjustment factor	Pre-splitting	10
***SMR** = RMR+(F_1xF_2xF_3)+F_4*	*46+{0.85x0.70x(-6)}+10*	***52.43***
10. Class	III	

200 joint attitudes were analyzed and two sets of joints deciphered from the pole (Fig. 6a) and contour diagrams (Fig. 6b) to cause instability to the slope, due to interference.

1. Slope : 70°→284°
2. J_1 : 54°→272°
3. J_2 : 53°→320°

The stereographic projection shows the development of a distinct wedge due to the intersection of these two joint sets. The true dips of both J_1 and J_2 lie outside the slide envelope (Fig. 6c). Cruden (1978) suggests double plane wedge failure in such cases. The rose diagram (Fig. 6d) suggests complex deformation, including shearing of the rocks.

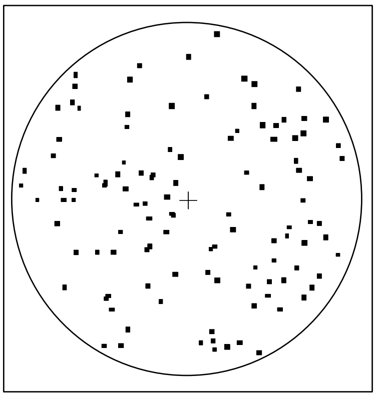

Fig. 6a. Pole diagram

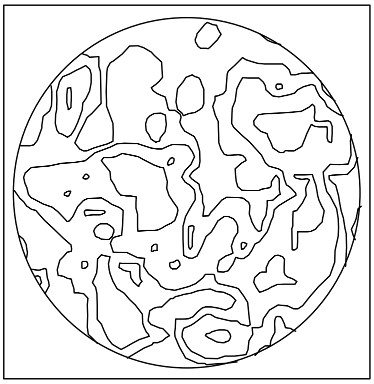

Fig. 6b. Contour diagram

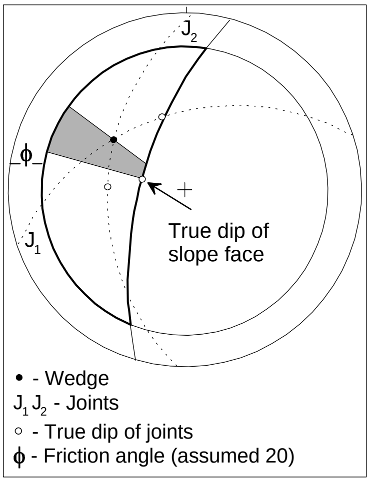

Fig. 6c. Stereogram

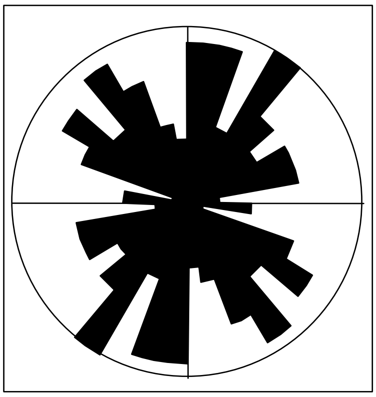

Fig. 6d. Rose diagram

Recommendations

a. The free face of the slope is steeply inclined hence, benching along a length of about 80 m is recommended to increase stability (Fig. 7).

b. A carpet of grass planted on the exposed surface will help hold the soil and prevent excess seepage of water into the soil.

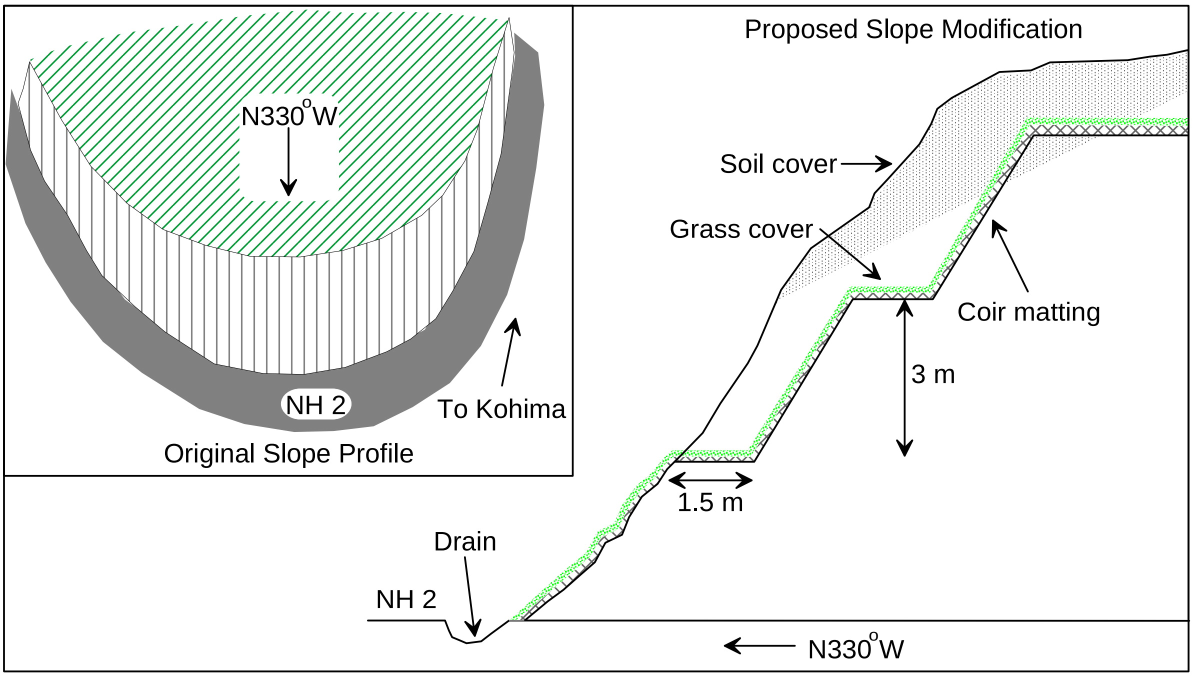

Fig. 7. Proposed slope modification / mitigation measures

Acknowledgement

A grant [No.NR-LS(NE-5)-07, dated 29.10.2007] provided by the DST is gratefully acknowledged.

References

Aier, I., Luirei, K., Bhakuni, S.S., Thong, G.T. and Kothyari, G.C. (2011). Geomorphic evolution of Medziphema intermontane basin and Quaternary deformation in the schuppen belt, Nagaland, NE India. Zeitsch. für Geomorph., vol. 55, pp. 247-265.

Aier, I., Singh, M.P., Thong, G.T. and Ibotombi, S. (2012). Instability analyses of Merhülietsa slide, Kohima, Nagaland. Nat. Haz., vol. 60, pp. 1347-1363.

Aier, I., Supongtemjen and Thong, G.T. (2009). Slope mass rating and kinematic analyses along part of NH 61, Nagaland, NE India. Intl. Jour. Earth Sci. Engg., vol. 2, pp. 520-526.

Barton, N.R. and Choubey, V. (1977). The shear strength of rock joints in theory and practice. Rock Mech., vol. 10, pp. 1-54.

Bhattacharjee, C.C. (1991). The Ophiolites of northeast India: a subduction zone Ophiolite Complex of Indo-Burman Orogenic Belt. Tectonophysics, vol. 191, pp. 213-222.

Bieniawski, Z.T. (1979). The geomechanic classification in rock engineering application. Proc. 4th Intl. Conf. Rock Mech. ISRM Moutreuse. Balkema, Rotherdam, vol. 2, pp. 51-58.

Chen, H. and Lee, C.F. (2002). A dynamic model for rainfall-induced landslides on natural slopes. Geomorphology, vol. 51, pp. 269-288.

Crozier, M.J. (1989). Landslide hazard in the Pacific islands. In: Brabb, E.E. and Harrod (Eds), Landslides: Extent and economic significance. Balkema, Rotterdam, pp. 357-366.

Cruden, D.M. (1978). Discussion of G. Hocking's paper: A method for distinguishing between single and double plane sliding of tetrahedral wedges. Intl. Jour. Rock Mech. Min. Sci. Geomech. Abstr., vol. 15, pp. 217.

Deere, D.U., Hendron, A.J., Patton, F.D., and Cording, E.J. (1967). Design of surface and near surface construction in rock. In: Fairhurst, C. (Ed.), Failure and breakage of rock. Proc. 8th US Symp. Rock Mech. New York. Soc. Min. Engg, Am. Inst. Min. Metall. Petrol. Engg. pp. 237-302.

Directorate of Geology & Mining, Nagaland (1978). Misc. Pub. No. 1.

Dortch, J.M., Own, L.A., Haneberg, W.C., Caffee, M.W., Dietsch, C., and Kamp, U. (2008). Nature and timing of large landslides in the Himalaya and Trans-Himalaya of northern India. Quat. Sc. Rev., doi:10.1016/j.quascirev.2008.05.002.

Emelyanova, E.P. (1977). The influence of geological and climatic conditions on the distribution of landslides on Russian Platform. Bull. IAGE., vol. 16, pp. 6-27.

Ghose, N.C., Agrawal, O.P., and Chatterjee, N. (2010). Geological and mineralogical study of eclogite and glaucophane schists in the Naga Hills Ophiolite, Northeast India. Island Arc, vol. 19, pp. 336-356.

Gray, D.H. (1973). Effects of forest clear-cutting on the stability of natural slopes: Results of field studies. Natl. Sc. Found., Washington, p. 119.

Hocking, G. (1976). A method for distinguishing between single and double plane sliding of tetrahedral wedges. Intl. Jour. Rock Mech. Min. Sci. Geomech. Abstr., vol. 13, pp. 225-226.

Hoek, E. and Bray, J.W. (1981). Rock slope engineering. Inst. Min. Metal., London.

Imchen, W., Thong, G.T. and Pongen, T. (2014). Provenance, tectonic setting and age of the sediments of the Upper Disang Formation in the Phek District, Nagaland. Jour. Asian Earth Sci., vol. 88, pp. 11-27.

Kliche, C.A. (1999). Rock slope stability. SME, Littleton Co.

Markland, J.T. (1972). A useful technique for estimating the stability of rock slopes when the rigid wedge sliding type of failure is expected. Imp. Col. Rock Mech. Res. Rep., vol. 19, p. 10.

Mathur, L.P. and Evans, P. (1964). Oil in India. 22nd Intl. Geol. Cong., N. Delhi.

Murthy, V.N.S. (2001). Principles of soil mechanics and foundation engineering (5th ed). UBS Publishers & Distributors Ltd., 842p.

Nandy, D.R. (1976). The Assam syntaxis of the Himalaya: A re-evaluation. Sem. Rec. Geol. Study, Him. Misc. Pub., Geol. Surv. India, vol. 24, pp. 363-368.

Nilsen, T.H., Taylor, F.A., and Brabb, E.E. (1976). Recent landslides in Alameda County, California. USGS Bull., No. 1398, pp. 1-21.

Nishida, Y., Yagi, N., and Futaki, M. (1979). The theoretical analysis of the pore pressure due to rain water permeation in the ground. Proc. 6th Asian Reg. Conf. Soil Mech. Found. Engg., Singapore, vol. 1, pp. 241-244.

Palmström, A. (1982). The volumetric joint count - a useful and simple measure of the degree of rock jointing. Proc. 4th Conf. Intl. Assoc. Engg. Geol., Delhi, vol. 5, pp. 221-228.

Petley, D.N. and Reid, S. (1999). Landscape sensitivity and change at Taroko, eastern Taiwan. In: Smith, B.J., Whalley, W.B., and Warke, P.A. (Eds), Spl. Publ., Geol. Soc. Lond., vol. 162, pp. 179-195.

Romana, M. (1985). New adjustment ratings for applications of Bieniawski classification of slopes. Intl. Symp. Rock Mech., Zacatecas, pp. 49-53.

Supongtemjen (2013). Geological investigation of land instability between Kohima and Zhadima. Unpublished PhD thesis, Nagaland University, Kohima.

Swanston, D.N. and Dyrness, C.T. (1975). Impact of clear cutting and road construction on soil erosion by landslides in the Western Cascade Range, Oregon. Geology, vol. 3, pp. 393-396.

Swanston, D.N. and Swanson, F.J. (1977). Timber harvesting, mass erosion and steep land forest geomorphology in the Pacific northwest. In: Coates, D.R. (Ed), Geomorphology and engineering. Hutchinson & Ross, Stroudsburg, pp. 199-221.

Terzaghi, K. (1950). Mechanism of landslides. In: Paige, S. (Ed), Application of geology to engineering practice. Geol. Soc. Am. Mem., pp. 83-123.

Towhata, I. (2007). New strategy and tools for mitigation of landslide disasters. In: Ayothiraman, R. and Hazarika, H. (Eds), Earthquake hazards and mitigation. Proc. Intl. Workshop Eqk. Haz. Mit., Guwahati, pp. 116-124. I.K. International Publishing House, New Delhi.

Valdiya, K.S. (1987). Environmental geology: Indian context. Tata-McGraw Hill, New Delhi, 583p.

Veder, C. and Hilbert, F. (1980). Landslides and their stabilization. ISBN, USA.

Verma, R.K. (1985). Gravity field, seimicity, and tectonics of the Indian Peninsula and the Himalaya. Allied Publishers, New Delhi, pp. 155-189.