RISK ANALYSES ALONG PART OF NH 2, NORTH OF KOHIMA TOWN, NAGALAND

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ABSTRACT

The NH 2, starting from the north of Kohima town, is being widened without any mitigation measures in place, resulting in numerous debris slides. In this connection four slides were studied to determine slope stability conditions and the potential and modes of probable failure, for which slope mass rating and kinematic analyses were carried out in and around the slide zones. Results indicate that the slopes are very unstable and prone to further sliding, induced by small planar or wedge failure of the rock masses.

SOCIETAL RELEVANCE

Fresh cut slopes of weak material in high rainfall regions are problematic, so remedial and/or mitigation measures should be in place during excavation. However, this is not so in the present study area. Therefore, risk analysis was carried out to determine the causes of the debris slides. Simple mitigation measures worked out will ensure that sliding is largely arrested and the life of the road is enhanced.

Keywords : Risk analyses, SMR, Kinematic analyses, NH 2, Kohima

INTRODUCTION

Development in hilly areas also includes widening of existing highways to cater to the needs of growing populations, which could promote instability. It is therefore necessary to comprehensively assess the factors that lead to failure. These may include topography, soils, lithology, structure, hydrology, rainfall, and human activity. The amount of shearing and fracturing and the attitude of beds or joints with respect to slope geometry are important criteria in determining slope stability conditions. The instability of slopes is also influenced by accumulations of debris in the head regions. Reduction in shear strength is usually attributed to high pore-water pressure and large slope deformations. Slopes fail when slope material progressive deteriorates [1] and the critical slope angle is exceeded. The slightest provocation at this stage, such as heavy rainfall, reduces the shearing strength of the slope forming material to cause failure.

Lithology and structure greatly influence landslide occurrences because of variations created in strength and permeability of rocks and soils. Weathering greatly reduces the shearing resistance of rocks. Planes of weakness within rock masses determine the stability of rock slopes to a great extent. Their physical and mechanical properties are a function of the attitude, geometry, and spatial distribution of these planes. Failure in rock masses tends to follow pre-existing discontinuities and do not occur throughout the intact rock to any great extent unless the rock is weak or incompetent. A soil mass is a relatively homogenous and continuous medium composed of loose particles. Failure in soil tends to occur within the soil mass and the direction of failure does not depend on variations of soil properties.

Geologic and tectonic structures greatly affect slopes. They are important when they occur along the slopes or ridges of local topography [2]. Faults and joints are the most prominent of geological discontinuities that affect slope stability. The seepage of water along joints, faults, and bedding planes is responsible for the occurrence of many slides. Increase in moisture content in joints filled with clay can cause considerable swelling pressure which may also lead to sliding.

Groundwater in hilly terrain is generally channelled along structural discontinuities of rocks or diffused through crushed rocks or loose soil masses. It is thus, difficult to evaluate its behaviour over large areas. In such terrain surface indications provide a quick appraisal of the nature groundwater with reference to stability of hill slopes.

Rapid urbanization with haphazard developmental activities is responsible for numerous landslides. Construction of highways without any scientific basis has also contributed to sliding of slope masses. Road cutting in areas of unfavourable dips of beds, and crushed and weathered rocks often disturb natural slopes and drainage configurations to bring down portions of hillsides.

Kohima, the capital city of Nagaland is connected to the other districts through a wide network of narrow highways. In recent years an important highway, the former State Highway 1, was upgraded and renamed National Highway (NH) 2. This highway connects Kohima to Jhanji in Assam. Widening of the highway has begun to that of four-lane category. However, slope excavations have been taken up very indiscriminately without any engineering considerations. This has led to slope failure at numerous places. Slopes commonly fail during the monsoon. The study area is made up dominantly of shale, much, of which are jointed, fractured, crumpled, and weathered. Water seepage into the subsurface is high during the monsoon. These waters seep out at various levels along hill slopes during this period.

Instability includes slumps and debris slides. Risk has been assessed in four weak portions along this highway. Rock and slope mass rating of slope material in the slide areas have been estimated and kinematic analyses performed to determine the probable modes of failure.

LOCATION

The study area, starting south of the Indoor Stadium, lies north of Kohima town (Figure 1). The NH 2 runs roughly N-S through this area. This area is part of the Survey of India topographic map no. 83 K/2.

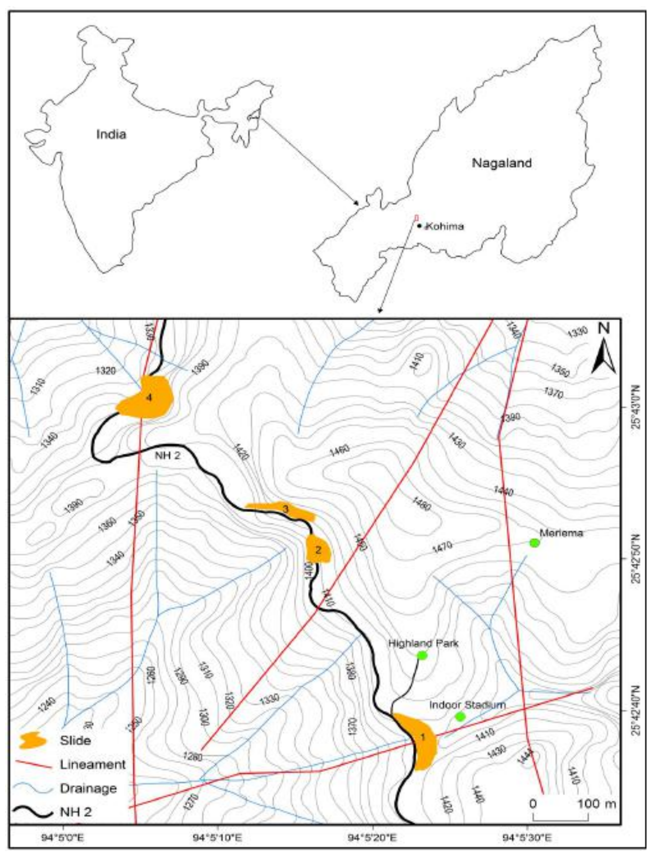

Figure 1. *Geological map of study area*

GEOLOGICAL SETTING

Nagaland is part of a major mobile belt of the earth's crust. The eastern margin of Nagaland represents the portion of the crust where the Indian Plate has subducted beneath that of the Burmese. This has compressed this geodynamically sensitive region, which has resulted in folding and faulting. The general trend of the lineaments is approximately NE-SW, indicating a NW-SE compression. This tectonically complicated and relatively young region comprises an immature, highly dissected mountainous terrain. Continuing tectonism is responsible for extensive jointing, shearing, fracturing, and crumpling of the rocks. Various geomorphic processes have further weathered and eroded the weakened rocks leading to large scale slope instability.

This part of the region is made up of the Disang and Barail groups of rocks of Upper Cretaceous-Eocene and Oligocene ages respectively. The Disang is dominantly made up of shales with minor intercalations of sandstones and siltstones. The Barail Group, occurring as outliers conformably over the Disang, is made up of thick bedded sandstones with alternations of thin beds of shales.

The study area is made up of denuded hills of the Disang Group. Towards the north the Barail form steeper ridges capping the Disang [3]. The shales of the Disang Group are highly crumpled and partially weathered at many places. Landslides frequently occur in the shale dominated areas. The Barail sandstones making up the ridge crests of this area are more or less stable, except where the basal Disang rocks have been removed by the natural processes. The soil cover in the area, including rock and soil debris, range in thickness from less than 3 m to more than 15 m. Soil exposures are commonly buff to pale gray.

METHODOLOGY

Studies of fresh-cut slopes enable an understanding of stability conditions. Four debris slides induced due to road widening were taken up for studies. Thirty to forty rock samples were collected from each location and analysed using a Point Load Index Tester. Averages of these samples were taken to determine their strengths.

The present study is based on the slope mass rating (SMR) of Romana [4] which is basically evolved from the rock mass rating (RMR), a rock mass classification of Bieniawski [5]. The RMR was computed by adding rating values for five parameters including strength of intact rock, rock quality designation, spacing and condition of discontinuities, and water inflow through discontinuities. The SMR was obtained from the RMR by adding an adjustment factor depending on the joint-slope relationship and adding a factor for the natural slope. Here, $SMR = RMR + (F_1xF_2xF_3) + F_4$.

Kinematic analyses have been performed along fresh-cut slopes of the highway. Here, angular relationships between discontinuities and slope surfaces were used to determine the potential and modes of failures after Kliche [6]. About 150 to 200 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. From the contour diagrams the dominant joint sets that control instability were identified. These were used to generate stereographic projections. Data generated from joint projections were used for kinematic analyses using the Markland's test [7], which was modified by Hocking [8], Cruden [9], and Hoek and Bray [10].

RESULTS AND DISCUSSION

For sustainable development in structurally disturbed hilly terrain, study of cut-slopes is of prime importance. 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 avert accidents.

The study area comprises Disang shale, where four debris slides have affected the highway. The region receives abundant rainfall during the monsoon. Storms and intense rainfall are common during the period. A number of structurally controlled streams are characterised by parallel and trellis drainage patterns. Most of these streams dry up during winter. However, some seepage ponds contain water even during the dry season. Much of the subsurface in the sheared and crumpled zones are saturated during the monsoon. During intense rainfall, a lot of water also flows as surface runoff. As fresh cut slopes have not been provided adequate roadside drainage uphill waters flow onto the highway. Continued debris slides have littered the road as well, leading to rapid deterioration of the asphalt. Four lineaments are noted in the study area. Two of these are NNE-SSW and WSW-ENE trending and two have approximate N-S trends. The NNE-SSW lineament, corresponding to the general compression direction of Nagaland, is the trend along which major thrusting in the region has taken place. The N-S lineaments correspond to strike-slip movements, along which antithetic shearing may have taken place. The WSW-ENE lineament may be a hybrid fracture.

Location 1 (6.45 km junction)

This small slide, lying at 25°42'39" N latitudes and 94°04'22.41" E longitudes, to the west of the Indoor Stadium, covers about 230 sq m. It has affected a 40 m stretch of the highway (Fig. 2). The slope is made up of sheared, jointed, and partially weathered shale. This is overlain by debris and young soils. The general trend of the slope ranges from 25° to 32° in a WSW direction. The rocks exhibit three prominent sets of joints. Widening of the road has left the weak slope, which is prone to continuous sliding, as high as 12 m with an average angle of 55º.

***Figure 2**. Debris slide in sheared rocks*

Slope mass rating and kinematic analyses

SMR values fall in Class IV, indicating unstable slope conditions (Table 1). From the contour diagrams of joint attitudes, three dominant joint sets are identified, which are plotted against slope in a stereographic projection (Figure 3).

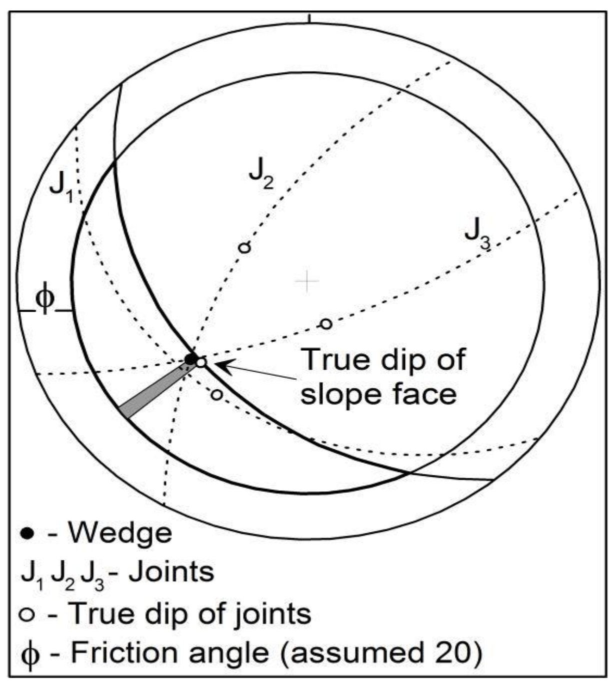

***Figure 3.** Stereographic projection (Location 1)*

Table 3. Slope mass rating (Location 3)

	Value or Condition	Rating
1. Point Load Test	1.45 MPa	4
2. RQD	42.4%	8
3. Spacing of joints	45.4 mm	5
4. Condition of joints	Slightly rough; separation <1mm; soft joint wall rock	12
5. Groundwater condition	Dry	10
RMR	*= (1+2+3+4+5)*	***39***
6. $F_1 = (1\text{-Sin}\lvert\alpha_j - \alpha_s\rvert)^2$	20°	0.70
7. $F_2 = \text{Tan}^2\beta_j$ or $F_2 = 1$ for toppling	40°	0.85
8. $F_3 = \lvert\beta_j - \beta_s\rvert$ for plane failure $= \lvert\beta_j + \beta_s\rvert$ for toppling where β_s= dip/angle of slope	-35°	-60
9. F_4 = Adjustment factor	Pre-splitting	10
SMR *= RMR+(F_1xF_2xF_3)+F_4*	*39+{0.70x0.85x(-60)}+10*	***13.3***
10. Class	V	
11. Description	Very unstable	

Table 4. Slope mass rating (Location 4)

	Value or Condition	Rating
1. Point Load Test	1.45 MPa	4
2. RQD	39.1%	8
3. Spacing of joints	43.4 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. $F_1 = (1\text{-Sin}\lvert\alpha_j - \alpha_s\rvert)^2$	10°	0.70
7. $F_2 = \text{Tan}^2\beta_j$ or $F_2 = 1$ for toppling	30°	0.40
8. $F_3 = \lvert\beta_j - \beta_s\rvert$ for plane failure $= \lvert\beta_j + \beta_s\rvert$ for toppling where β_s= dip/angle of slope	-45°	-60
9. F_4 = Adjustment factor	Pre-splitting	10
SMR *= RMR+(F_1xF_2xF_3)+F_4*	*39+{0.70x0.40x(-60)}+10*	***32.2***
10. Class	IV	
11. Description	Unstable	

Slope : 50°230°
J_1 : 45°217°
J_2 : 69°300°
J_3 : 76°159°

The true dips of the joint planes J_2 and J_3 lie outside the shaded region. The intersections of these joints indicate that the rocks may fail as small wedges. J_1 plotted against slope shows planar mode of failure. A hybrid fracture is responsible for shearing the rocks, thereby rendering the slope prone to sliding.

Location 2 (7.12 km junction)

This debris slide lies at 25°42'50.30" N latitudes and 94°05'16.94" E longitudes. The rocks are jointed and fractured. They comprise shale and minor siltstone with black and brown clay. The wide gaps in the fractures and joints are filled with clay. Rock dips are variable due to local structural disturbances. Three prominent sets of joints are noted. The upper slopes are covered by vegetation (Figure 4). The slope, varying in height from 6 to 15 m, has been left at an angle of about 80°.

Figure 4. *Debris slide in crushed zone*

Slope mass rating and kinematic analyses

RMR values indicate weak rocks and SMR values indicate unstable slope conditions (Table 2). From joint analyses three dominant joint sets are identified, which are plotted against slope attitude in a stereographic projection (Figure 5).

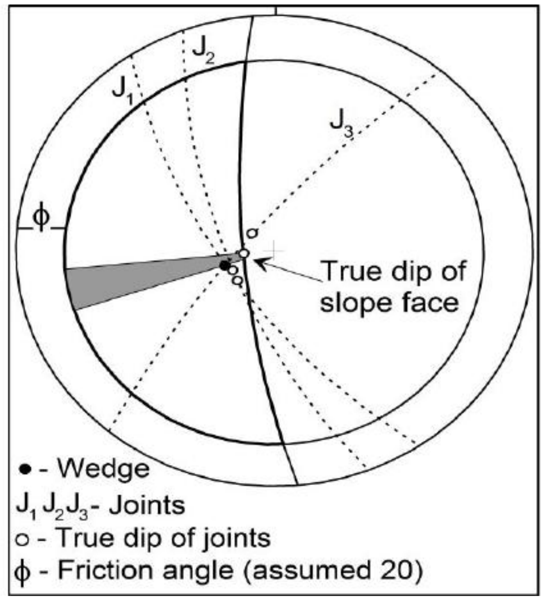

Figure 5. *Stereographic projection (Location 2)*

Slope : 80° 265°
J_1 : 74°236°
J_2 : 75°249°
J_3 : 81°310°

The true dips of the joint planes J_1 and J_3 lies outside the shaded region between the true dip of the slope and the line of intersection of the two joint planes (J_2 and J_3). J_2 plotted against slope indicates probable planar failure. Joint intersections (J_2 and J_3) suggest that the rocks may further break into small blocks.

Location 3 (7.30 km junction)

The slope is very high and steep at this location, which lies at 25°42'54.01" N latitudes and 94°51'13.28" E longitudes. The slope ranges from 15 to 20 m in height with an average inclination of about 75°. Debris slides have affected about 120 m of the road (Figure 6). The folded shale intercalated with minor siltstones are jointed and fractured. The bedrocks dip 70° towards 55°NE. The rocks exhibit three prominent sets of joints. The first set of joints trends WNW-ESE with moderate dips towards SSW. The second set trends NW-SE with almost vertical dips towards SW. The third set trends NNE-SSW with near vertical dips towards the SE.

Figuer 6. *Debris slide in fractured rocks*

Slope mass rating and kinematic analyses

RMR values indicate weak rocks and SMR values of Class V indicate very unstable slope conditions (Table 3). J_1 (63°188°) is identified as the dominant joint which is plotted against slope attitude (75°200°) in a stereographic projection (Fig. 7). The true dip of the joint plane J_1 lies at an angle of ±12° with respect to the true dip of the slope face. Such a relationship may cause planar failure.

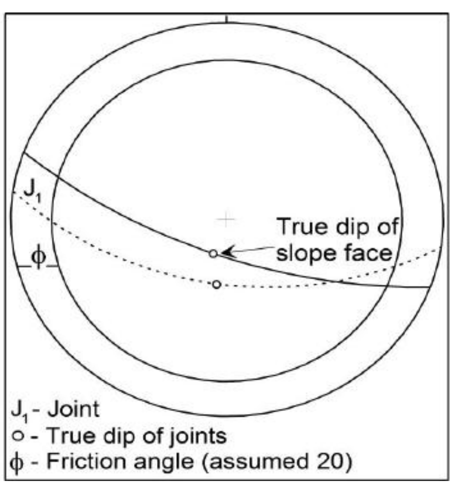

Figure 7. *Stereographic projection (Location 3)*

Location 4 (7.80 km junction)

This section is located at 25°42'59.15" N latitudes and 94°05'5.50" E longitudes. A debris slide damaged a 50 m stretch of the highway (Figure 8). The rocks are highly crushed and weathered. Three prominent sets of joints are noted.

The crumbled and weathered shale are capped by loose debris on a 75° slope at this location. The height of the slope is approximately 10 m. Water enters the joints and loose debris to saturate the slope material. The upper portion of the area is clad with trees planted a couple of years back, which add to the load on the weak slope.

Figure 8. *Debris slide in crushed zone*

Slope mass rating and kinematic analyses

RMR values indicate the presence of weak rocks while SMR values indicate unstable slope conditions (Table 4). Joint data are plotted against slope attitude in a stereographic projection (Figure9).

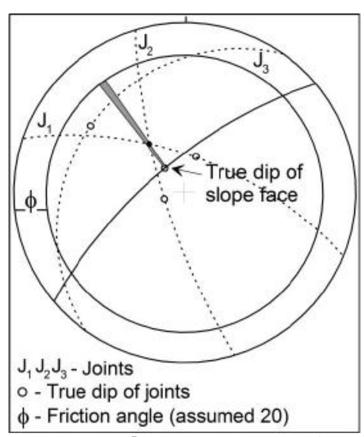

Figure 9. *Stereographic projection (Location 4)*

Table 3. Slope mass rating (Location 3)

	Value or Condition	Rating
1. Point Load Test	1.45 MPa	4
2. RQD	42.4%	8
3. Spacing of joints	45.4 mm	5
4. Condition of joints	Slightly rough; separation <1mm; soft joint wall rock	12
5. Groundwater condition	Dry	10
RMR	*= (1+2+3+4+5)*	***39***
6. $F_1 = (1\text{-Sin}I\alpha_j - \alpha_s I)^2$	20°	0.70
7. $F_2 = \text{Tan}^2\beta_j$ or $F_2 = 1$ for toppling	40°	0.85
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	-35°	-60
9. F_4 = Adjustment factor	Pre-splitting	10
SMR $= RMR+(F_1xF_2xF_3)+F_4$	*39+{0.70x0.85x(-60)}+10*	***13.3***
10. Class	V	
11. Description	Very unstable	

Table 4. Slope mass rating (Location 4)

	Value or Condition	Rating
1. Point Load Test	1.45 MPa	4
2. RQD	39.1%	8
3. Spacing of joints	43.4 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. $F_1 = (1\text{-Sin}I\alpha_j - \alpha_s I)^2$	10°	0.70
7. $F_2 = \text{Tan}^2\beta_j$ or $F_2 = 1$ for toppling	30°	0.40
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	-45°	-60
9. F_4 = Adjustment factor	Pre-splitting	10
SMR $= RMR+(F_1xF_2xF_3)+F_4$	*39+{0.70x0.40x(-60)}+10*	***32.2***
10. Class	IV	
11. Description	Unstable	

Slope : 75° 320°
J_1 : 70°20°
J_2 : 80°253°
J_3 : 33°305°

The true dips of the joint planes J_1 and J_2 lie outside the shaded region. The area is prone to double plane wedge and planar failure. Shearing effects of the lineament has helped crush the rocks.

A rose diagram of joints of the region (Figure 10) shows concentration along NW-SE. These joints are due to tensile stresses that are concentrated around the regional NW-SE compression. Another set of joints trend NW-SE which is parallel to the regional trend. As a consequence hybrid fractures have developed in the terrain due to the interplay of these stresses causing extensive deformation of the rocks. Jointing and fracturing have considerably weakened the rock masses thereby initiating extensive weathering to produce thick mantles of waste, making the area susceptible to sliding, particularly during the monsoon.

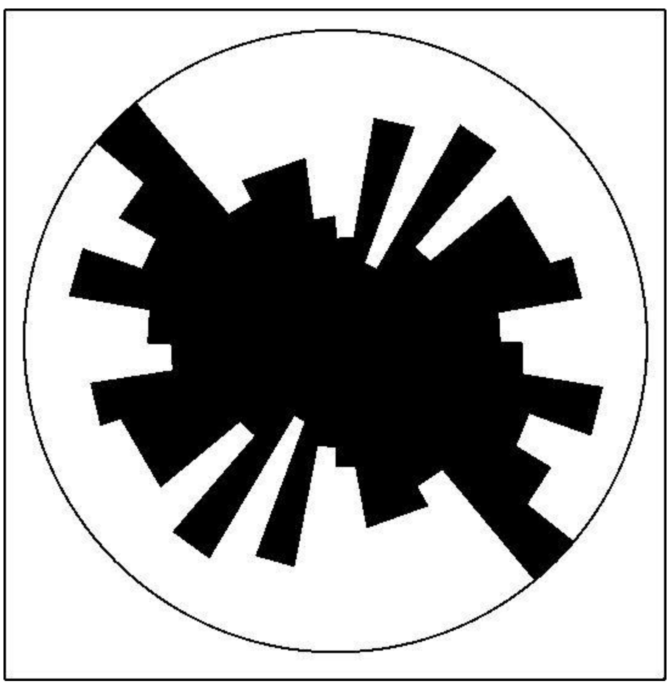

Figure 10. *Rose diagram of joints*

The rocks are affected by numerous, closely spaced joints. These joints are so oriented that failure is mostly planar, but small wedges too are noted. However, due to probable ongoing tectonism (Aier et al., 2012)[11], shearing and squeezing has led to fracturing of the rocks. Clays deposited within these fractures and abundant joints start to swell in the presence of moisture during the monsoon. This undesirable pressure causes the rocks to crumble, thereby rendering them soil-like. With abundant water and weathered material in the form of clays at these sites, conditions are ideal for debris slides even in the future.

CONCLUSIONS

Road making techniques are very poor in the state. The debris from these slopes flowing down onto roads erodes the bitumen rapidly, which proves to be a costly affair in road maintenance. Hence, it should be seriously considered to implement mitigation measures that have been provided. Other parts of NH 2 beyond the study area that are weak should also be considered for similar appropriate measures. Detailed geotechnical analyses for appropriate mitigation and/or remedial measures need to be taken up in areas proposed for urban expansion in the highly susceptible zones.

Fresh cut slopes along the roads are apparently stable at the moment. However, at a number of places they can create havoc if left unattended. Under such circumstances top soil or parts of slope should be removed to reduce the driving forces. Surface water is one of the major factors that cause landslides. Thus, appropriate drainage facilities should be provided where slope material is susceptible to erosion, particularly under unfavourable groundwater conditions. As far as possible, water should not be allowed to enter landslide zones, old or new. Surface drainage may be necessary around crowns of present slides to prevent sheet wash from entering landslide areas. It is also necessary to remove excess water from the subsurface so as to reduce pore-water pressure below which can cause slope failure.

RECOMMENDATIONS

Roadside drains and favorably modified slopes or slopes with adequate lateral support are important.

Location 1: Surface runoff along the slide zones should be channelized effectively by a good roadside drain. An appropriately designed retaining wall along the road is necessary to arrest the slope mass.

Location 2: The present retaining should be built higher and requires to be extended through the entire length of the affected area.

Location 3: The vertical exposed rock surface should be bound with GI wire to prevent debris falls. Regular maintenance is necessary.

Location 4: Appropriately designed, higher retaining walls are necessary. Trees from the upper slope should be removed.

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