GEOTECHNICAL STUDIES OF THE VISWEMA SLIDE, KOHIMA, NAGALAND

Meniele Khalo[1], Soibam Ibotombi[2], Supongtemjen[3], Glenn T. Thong[4*]
[1]Department of Geology, Kohima Science College, Jotsoma
[2]Department of Earth Sciences, Manipur University, Canchipur, Imphal
[3]GIS & RS Center, Planning & Coordination Department, Govt. of Nagaland, Kohima
[4]Department of Geology, Nagaland University, Kohima Campus, Meriema
*Corresponding Author: glen2t03@yahoo.com

ABSTRACT

The present study is an attempt at slope stability determination using slope mass rating and kinematic and geotechnical analyses. Prolonged rainfall during the monsoon led to saturation of the slope material and triggered a massive landslide along the Asian Highway 1 during the last week of August 2007. The area is still affected by landslides Analysis of consistency limits of soils from the landslide area indicates that these soils are highly saturated during the rainy season and exceed the plastic limit and reach the liquid limit. Such a situation commonly triggers landslides.

SOCIETAL RELEVANCE

Regions that receive heavy rainfall and possess weak slope material are prone to landslides, particularly those areas where problematic slopes have been excavated for development but without any remedial/mitigation measures in place. Risk analysis was carried out in the study area to determine the causes of the debris slide and based on which simple mitigation measures have been worked out to ensure that sliding is largely arrested and the road made pliable.

Keywords: AH 1, SMR, Kinematic analyses, Geotechnical analyses

INTRODUCTION

Nagaland is a hilly state situated in the far northeastern corner of India. It is part of the Indo-Myanmar Range. It represents a narrow strip of mountainous country with Manipur to its south. Nagaland occupies an area of 16,579 sq km. Of the total geographical area, about 90% is rugged hilly terrain with thick deciduous forests.

The region is a tectonically disturbed young mobile belt. As a result the rocks are highly folded, jointed, faulted, sheared, and fractured. The tectonic disturbance is related to the subduction of the Indian Plate beneath the Burma Plate, which resulted in over-thrusting of the older formations above the younger ones. The older formations generally occupy the eastern part of the state while younger formations lie towards the west [1].

Landslides are common natural hazards in young mobile belts. The Asian Highway (AH) 1, the lifeline between Nagaland and Manipur, is often affected by numerous landslides and subsidence, being very weak at numerous places (Anand [2] and Aier et al. [3]). It consists of lithounits of the Disang (Eocene) and Barail (Oligocene) groups. The Barail are made up of sandstone interbedded with thin shale while the Disang are composed of shale with thin beds of sandstone.

Parallel and angular stream patterns are common around the study area, their courses being controlled by joints and fault planes. A fault of local extent is also noted here. Shear zones and areas of crushed rocks form the major sites of sinking and sliding. The rocks are affected by 3 to 4 sets of joints, two being important. The shear zones are responsible for continuous subsidence of the highway at places during the monsoon. The study area is part of the Survey of India topographic map no. 83 K/2. It lies between 25°33'38.76" & 25°34'02.29" N latitudes and 94°08'22.8" & 94°08'38.97" E longitudes (Figure 1).

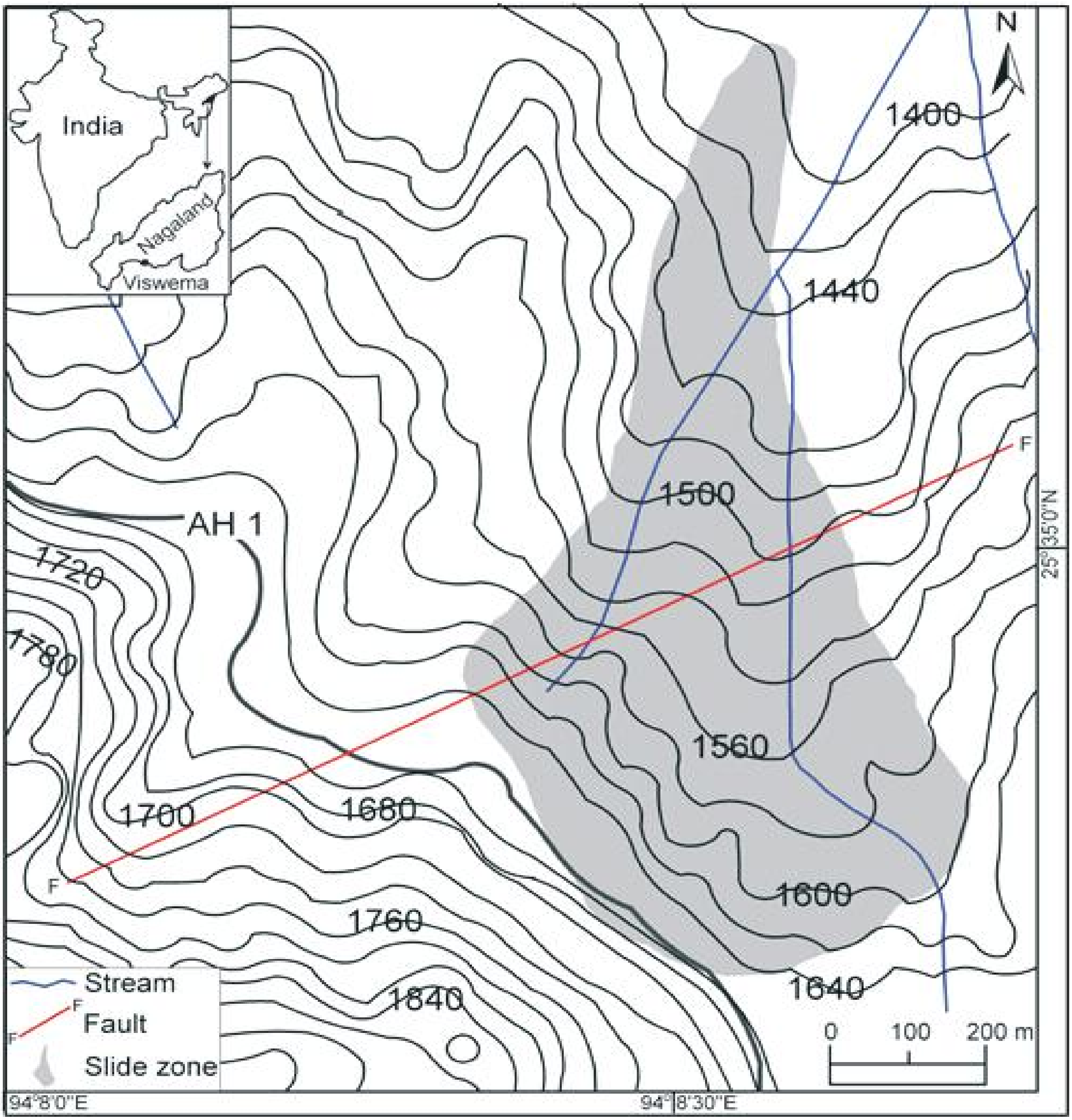

Figure 1. Geological map of study area

Studies were carried out to ascertain the factors responsible for instability. This is a very unstable area that was affected from the early 1960's and has continued to pose problems till today (Figure 2). Highway blockage in the past often disrupted communication between Kohima and Imphal. The length of the active slide zone is about 1200 m while the breadth is about 440 m, covering an area of 408139.48 sq m. Maximum damage to the highway included a length of 120 m, which kept the Border Roads Organisation constantly busy to keep it functional. Retaining walls are continuously constructed but they did not last long as their foundations are constructed on slide debris. Moreover, the heavy foundations exert additional burden on the soil mass. The drains constructed were shallow and ill maintained. Hence, abundant water was allowed to percolate into the subsurface. Change in water content can affect the stability of slope material and have been responsible for triggering, reinitiating, and accelerating more landslides than any other factor [4]. Meusburger and Alewell [5] claim that enhanced anthropogenic activity and adverse environmental factors lead to increase in landslide activity.

Figure 2. *Damaged highway*

GEOLOGY AND STRUCTURE

Bedrocks are not exposed in the affected area but nearby outcrops show the presence of Barail sandstone with thin beds of shale. The area consists of abundant debris including black and brown shale and clay. The highly sheared sandstones exposed are brown in color whereas the shales are dark. These rocks exhibit two prominent sets of joints, 50°135° and 45°350°. A NE-SW trending lineament cuts through across the area.

CAUSES AND EFFECTS

Studies have revealed that a number of factors are responsible for the failure of the slope. The sandstones are sheared to a high degree making them friable and weak and highly susceptible to weathering. The shales have been weathered to fine clay. The intense fracturing and crumpling of the rocks of this area may be due to ongoing tectonic movements. The black and brown clays are very weak and in the liquid limit state. The area received abundant rainfall of 416.5 mm in the month of August, 2007 (Table 1). Starting from April there was a gradual build-up in the amount of rainfall. This led to saturation of the ground. Waters that percolated into the subsurface were trapped in the poorly permeable clays in the lower slopes where terrace cultivation is practised. This probably caused the expansion of the clay minerals in the shale, leading to water logging. This in turn put tremendous pressure on the soils by increasing the pore pressure. These factors together had caused a large section of the road to be washed away along with the terrace fields. Loss of frictional resistance due to excess moisture led to reduction of shearing strength, which resulted in the devastation. The area remains damp and marshy throughout the year due to a very shallow water table. On the lower reaches of the slope a road leads to the village located immediately below the highway. The slope had been cut and modified for construction of road but it needs careful monitoring as it is also severely affected. Part of this village road too was affected.

Table 1. Rainfall data (mm) from April to August 2007

Month	April	May	June	July	August
Average	146.5	343.5	241.5	314.5	416.5

Source: Directorate of Soil & Water Conservation, Nagaland, Kohima

Slope mass rating

Forty five rock samples were collected from the site to determine their strengths. Point load test data [6] of fresh rocks indicates low value for the rocks. RMR [7] values of 34 indicate fairly stable rocks. SMR [8] values fall in Class IV, which indicates partially stable slope conditions that requires systematic maintenance.

Kinematic analyses

Kinematic analyses following Markland [9], Goodman [10], Hocking [11], Cruden [12], Lucas [13], Hoek and Bray [14], Matherson [15], and Yoon et al. [16] were performed from 200 joint attitudes taken in the field to determine the probable mode of failure. From joint analyses, pole density (Figure 3a) and contour diagrams (Figure 3b) were constructed and from which two dominant joint sets were identified. These were plotted against slope attitude in a stereographic projection (Figure 3c).

J_1 : 50°135°

J_2 : 45°350°

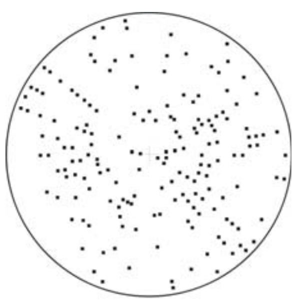

Figure 3a. *Pole diagram*

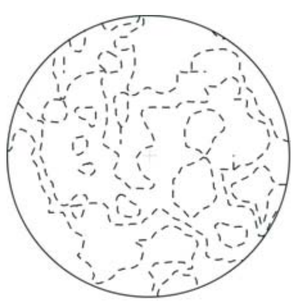

Figure 3b. *Contour diagram*

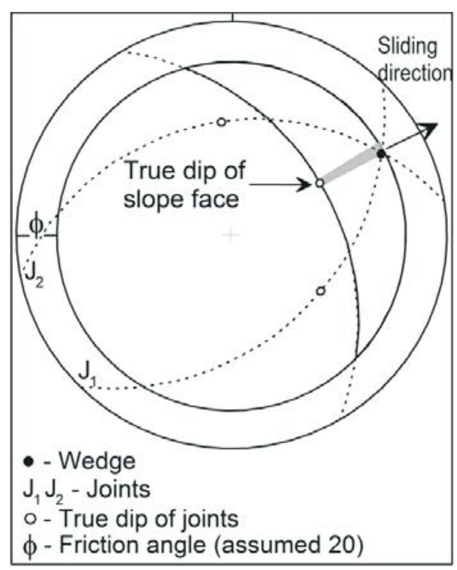

Figure 3c. *Stereogram*

The diagram shows a distinct wedge due to the intersection of joints J_1 and J_2. Cruden [12] suggests double-plane wedge failure in such cases as the true dips of both joints lie outside the shaded area. A rose diagram of this area (Figure 3d) shows that the lineaments are in conformity with that of the general trend of the region that is affected by F_1 folds. The rocks are thrusted along this plane. Antithetic and synthetic shearing stresses have also played a role in rock deformation.

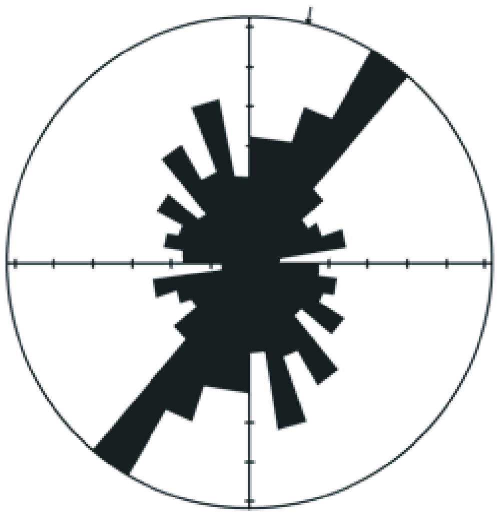

Figure 3d. *Rosette*

Geotechnical analysis of soils

Soil samples (V_1, V_2, V_3) were collected from the affected area (Table 2). One soil sample each was collected from the adjoining stable area (V_4). A series of laboratory tests were carried out to determine their geotechnical properties.

Sample No.	Natural water content	Plastic Limit (W_P) %	Liquid Limit (W_L) %	Plasticity Index (I_P) %	*Liquidity Index (I_L) %	*Consistency Index (I_C) %	Flow Index (I_f) %	Toughness Index (I_T) %	Shrinkage Limit (S_L) %	Shrinkage Ratio (SR) %	Volumetric Shrinkage (V_S) %
V_1	26.42	28.83	46.50	17.67	1.00	0.00	23.50	0.75	16.58	1.80	76.48
V_2	46.43	19.75	38.00	18.25	1.46	-0.46	10.00	1.83	8.73	2.10	84.71
V_3	17.99	20.89	39.50	18.61	1.37	-0.37	14.00	1.33	12.80	1.91	71.79
V_4	19.74	22.90	56.50	33.60	0.70	0.30	26.00	1.29	16.92	1.76	76.94

*Values calculated with highest natural water content

Liquid limit (W_L)	46.50	38.00	39.50	56.50
Plasticity index (I_p)	17.67	18.25	18.61	33.60

Consistency limits of the slope material were determined to understand the firmness of the soils. A plasticity chart (Figure 4) prepared for the soils indicate that soil samples falls under OI group, which is indicative of organic silts of medium plasticity and between CI and MI group, depicting that the soils are inorganic clays of moderate plasticity and inorganic silts of moderate plasticity.

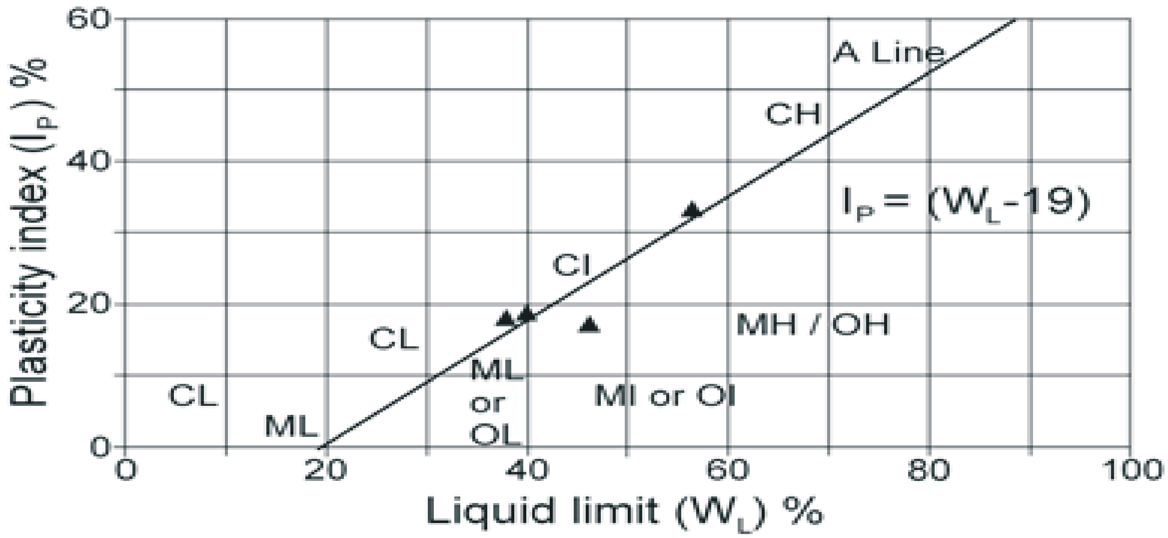

Figure 4. *Plasticity chart (IS: 1498-1970)*

Consistency index values computed from the maximum water content show values less than 0 (negative) indicating that failure took place during the liquid stage. The moisture content of the soils is greater than the liquid limit, which suggests that the soils were already in a liquid state at failure. Since data was collected after the rainy season, values of the maximum moisture content have been considered. Therefore, with or without additional water in the area the zone is bound to give way. Hence, it is expected that these slopes will fail anytime of the year however, becoming more unstable during prolonged rains. Liquidity indices of these soils are more than zero indicating that the zone is in an unstable state.

Direct shear test

The samples (V_1-V_4) were subjected to direct shear tests under different values of normal stresses. The results obtained for these tests are shown in Table 3.

Table 3. Normal and shear stresses and other parameters

Sample	σ_n (normal stress) kN/m^2	τ (shear stress) kN/m^2	Stress ratio τ/σ	Water content %
V_1	9.8	10.83	1.11	26.80
V_2	19.6	12.78	0.65	20.89
V_3	29.4	15.27	0.52	26.81
V_4	39.2	16.11	0.41	21.19

Cohesion (C) = 8.5kN/m^2; Internal friction angle (φ) = 13°

Normal ($_n$) and shear stresses () are plotted against each other (Figure 5). A best fit line gives the Coulomb failure equation, =c+$_n$tan. The cohesion and internal friction angle of the soil determined are 8.54 kN/m^2 and 13 respectively.

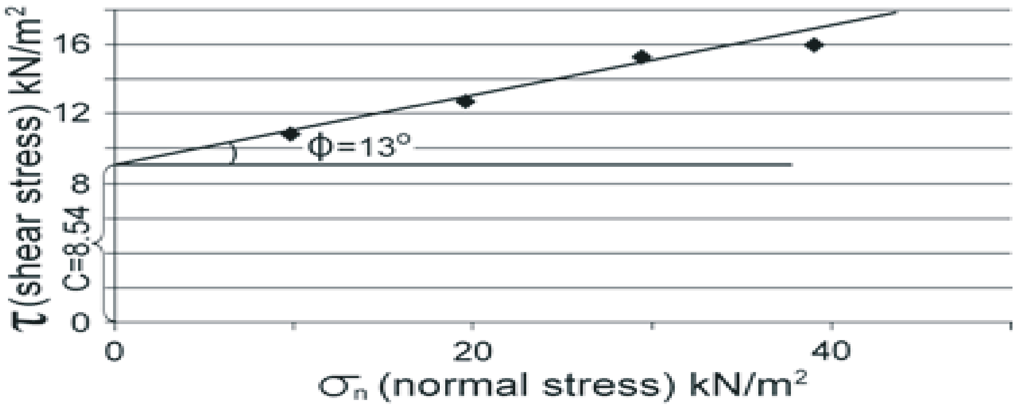

***Figure 5.** Plots of normal and shear stresses*

The stress and strain, i.e., load and displacement curves of the four samples were plotted to determine the geotechnical nature and behavior of these soils (Figure 6). From the graphs it appears that these soils are mainly clay with some amount of silt and sand; all the samples have good amount of clay except V_1 which has some amount of silt and sand. As the cohesive strengths as well as internal friction angles are low, these could be mostly clay with a little silt and fine sand. The value of cohesion (c) is 8.54 kN/m^2 while the internal friction angle is 13. Such slopes cut more than the internal friction angle are virtually unstable.

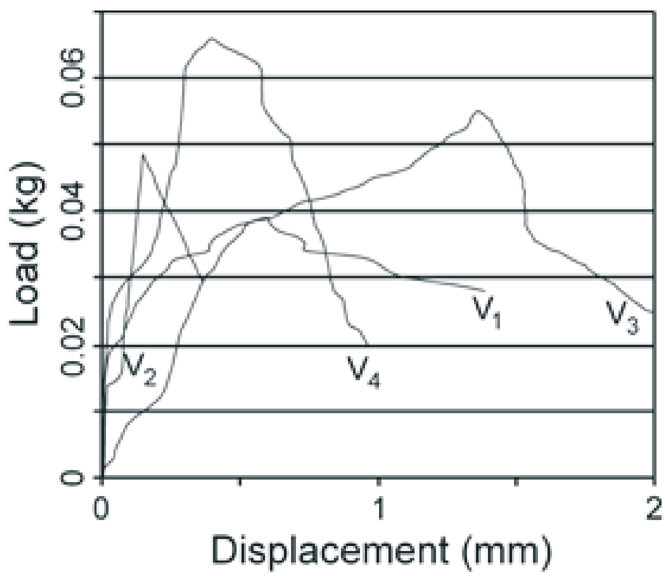

Figure 6. Stress / strain curves of the samples

RECOMMENDATIONS

1. Logically, marginal stabilization might be the best solution for this slide zone because of the large size and high cost of standard stabilization. A lower 'margin of stability' could be considered in an attempt to reduce the hazard level. If a "marginal stabilization" approach is adopted, mitigation measures could be applied in phases until the desired reduction in movement is accomplished.
2. A counterfort (Fig. 7a,b,c) with a height of 3 m and length of 30 m along a section of the road, with the foundation on bedrock is necessary.
3. Drainage of subsurface water through perforated pipes is necessary to prevent buildup of pore pressure.
4. A deep roadside drain with the road tilting towards the drain is vital to prevent water logging.
5. Drain water should be trained away from the slide zone.
6. A short term and low cost alternative is hydro-seeding the slope, a process which utilizes a slurry of seed to further prevent erosion and control subsurface seepage. This slurry sprayed over the prepared surface of the slide zone will help in the germination of plants and thereby stabilize the area to some extent.
7. *Vetiver* grass and hedgerows are effective in tropical and subtropical areas because of their fast growth and deep root penetration.

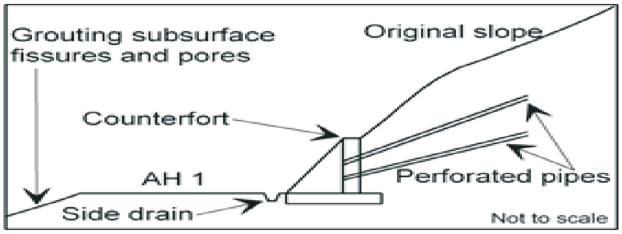

Figure 7a. *Proposed counterfort*

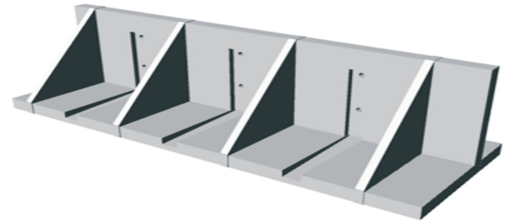

Figure 7c. *3D model of proposed counterfort*

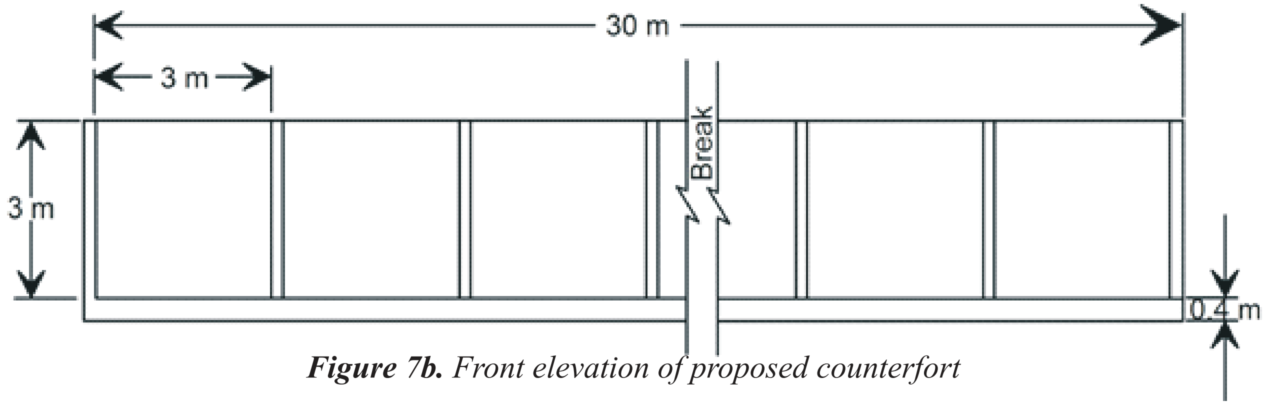

Figure 7b. *Front elevation of proposed counterfort*

REFERENCES

1. Ibotombi, S., 1998. Structural and tectonic analysis of Manipur with special reference to evolution of the Imphal Valley. Unpublished PhD thesis, Manipur University, Imphal, 283p.
2. Anand, R., 1988. Preliminary geological investigations of landslides along Dimapur-Mao road, Nagaland State, India. Ind. Geol. Assoc. Bull., vol. 21, pp. 199-205.
3. Aier, I., Pradipchandra, M., Thong, G.T., Soibam, I., 2012. Instability analyses of Merhülietsa slide, Kohima, Nagaland. Nat. Haz., vol. 60, pp. 1347-1363.
4. Crozier, M.J., 1986. Landslide causes, consequences and environment. Croom Helm Ltd., 535p.
5. Meusburger, K. and Alewell, C., 2008. Impacts of anthropogenic and environmental factors on the occurrence of shallow landslides in an Alpine catchment (Urseren Valley, Switzerland). Nat. Haz. Earth Sys. Sc., vol. 8, pp. 509-520.
6. Broch, E. and Franklin, J.A., 1972. Point load strength tlest. Int. Jour. Rock Mech. Min. Sci. Geomech, vol. 9, pp. 669-697.
7. Romana, M., 1985. New adjustment ratings for applications of Bieniawski classification of slopes. Intl. Symp. Rock Mech., Zacatecas, pp. 49-53.
8. Bieniawski, Z.T., 1979. The geomechanic classification in rock engineering application. Proc.4th Intl. Cong. Rock Mech. ISRM Moutreuse Balkema. Rotherdam, vol 2, pp. 51-58.
9. Markland, J.T., 1972. A useful technique for estimating the stability of rock slopes when the rigid wedge sliding type of failure is expected. Intl. Col. Rock Mech. Res. Rep., vol. 19, p. 10.
10. Goodman, R.E., 1976. Methods of geological engineering in discontinuous rocks. West Publishing, San Francisco.
11. 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.
12. 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, p. 217.
13. Lucas, J.M., 1980. A general stereographic method for determining possible mode of failure of any tetrahedral rock wedge. Intl. Jour. Rock Mech. Min. Sci. Geomech. Abstr., vol. 17, pp. 57-61.
14. Hoek, E. and Bray, J.W., 1981. Rock slope engineering (3rd ed). Inst. Min. Metal., London.
15. Matherson, G.D., 1988. The collection and use of field discontinuity data in rock slope design. Quart. Jour. Engg. Geology, vol. 22, pp. 19-30.
16. Yoon, W.S., Jeong, U., Choi, J.H., Kim, W.Y and Kim, C.S., 2002. Slope failure index system based on the behavior characteristics, SFi - System. Jour. Korean Geotech. Soc., vol. 18, pp. 23-37.