Landslides Management along National Highway-37, Manipur, Northeast India

Kh Mohon Singh

Department of Geology, Imphal College, Imphal, Manipur

Abstract

Landslides and related natural disaster frequently disturb the hilly routes of Manipur, especially the national highways of the state during and after heavy and prolonged rainfall. There are three important national highways in Manipur viz. NH-39, NH-37 and NH-150. NH-39 and 37 connect the state with other parts of the country acting as lifeline of the state. It causes extensive damages to roads, bridges, human dwellings, agriculture lands, forests and other establishments year after year resulting in immense losses of life and properties. The landslides in Manipur are classified into two broad groups based on their spatial locations i.e. landslides that affect the settlement areas and landslides that do not directly affect any habitation. In Manipur, dimension-wise, the larger landslides that are mostly the product of both natural and anthropogenic activities do cause inconvenience to the society but rarely lead to any fatality while the smaller landslides occur in plenty, mainly caused by anthropogenic activities and lead to fatality frequently.

A frequent landslide on Imphal-Jiribam National Highway-37 is a major obstruction for the free flow of traffic on it in all weathers. This highway has been affected by number of slides both in the rock masses as well as in the overburden materials which is a cause of great concern considering the safety of the life and properties. The steep and highly jointed slope along the road makes the zone prone to failure due to gravity and rainwater action. For the present study uniaxial compression strength test and Brazilian test have been employed in order to assess the mechanical strength of Miocene Surma sandstone of the slide area. The present study leads to the conclusion that rapid pace of anthropogenic interferences on the natural slope and prevailing unfavourable geological discontinuities along with rain water action have played an important role in triggering the rock fall. From the SMR study, the rock falls under class III of partially unstable category. The kinematic analysis and field observation suggest wedge failure.

Key words: Miocene Surma sandstone, uniaxial compressive strength, Brazilian test, kinematic analysis.

Introduction

Manipur is a small state located in a hilly terrain of Northeast India with a small portion of valley in the central part of the state and has a monsoon climate confined within four summer months from June to September. The southwest monsoon is the main source of rain, and June is the rainiest month. There are three seasons in the area- winter, summer and rainy season, though rainy season, as in the rest of India, coincides with summer months. Landslides and related natural disaster frequently disturb the hilly routes of Manipur, especially the national highways of the state during and after heavy and prolonged rainfall. NH-39 and NH-37 connect the state with other parts of the country acting as lifeline of the state.

Any constraint in these two highways affects the people of the state in many ways and dimensions. One of the main factors contributing to frequent landslides in Manipur is the fragile nature of the litho units, resulted from the rock types and structures characterizing these rocks. However, certain places are found to be covered with thick columns of soil rendering instability conditions. Engineering properties of rocks as well as soils and variation in moisture contents play major role in initializing slides on these slopes. So, in this paper we try to analyze the geotechnical properties of the rocks as well as the soils on the slope at Nung Dolan situated along the National Highway (NH-37) with a view to work out the instability conditions of the slope masses. The present investigation area falls under Survey of India (SoI) toposheet No. 83 H/5 at 24º46´8.2˝N & 93º18´1.3˝E. Geological formations exposed in the study area are highly fragile, jointed and structurally complex, belonging to the Surma group which is mainly made up of sandstone and shale with minor conglomerate at places.

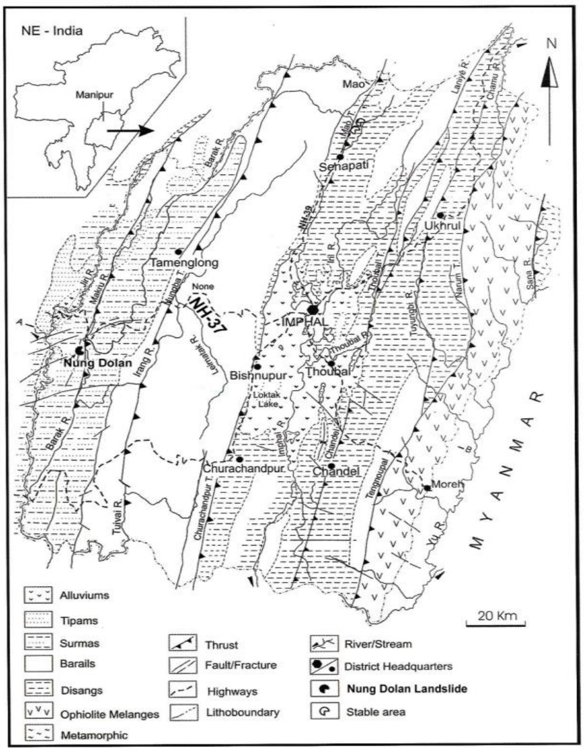

Figure 1: Geological map of Manipur (modified after Soibam, 1998) showing Nung Dolan Landslide (Source: Earth Sciences Deptt., Manipur University).

The entire study area is the hilly terrain. The slope of the landslide area is steep. The angle of the slope is an important factor in matters related to its stability. The steeper the slope, the greater is the chance of it becoming unstable. Blasting for road-cutting creates oscillations of a different frequency in the rocks, in the underground drainage system and in the shear strength of the slope materials in the area. These developments affect the forces operating on the slopes. Understandably, such activities tend to accelerate the process of slope failure by widening joints, fractures, fissures, and reducing cohesion of the slope material. In addition the entry of water through openings, fissures, fractures, joints, etc, reduces the shear strength of the rocks/soils as a result of pore-water pressure. With resistance to sliding thus lessened, the vulnerable rock mass falls and gives rise to landslides during or after heavy and prolonged rainfall due to gravity. Human activities that impede surface flow and disrupt underground circulation of water are also responsible for the building up of pore-water pressure, leading eventually to landslides. In the present study an effort has been made to work out the mode of failures that causes the instability of slopes at Nung Dolan. In present study the Landsat data is used because it is available since 1970s upto the present, free and easy online access through website.

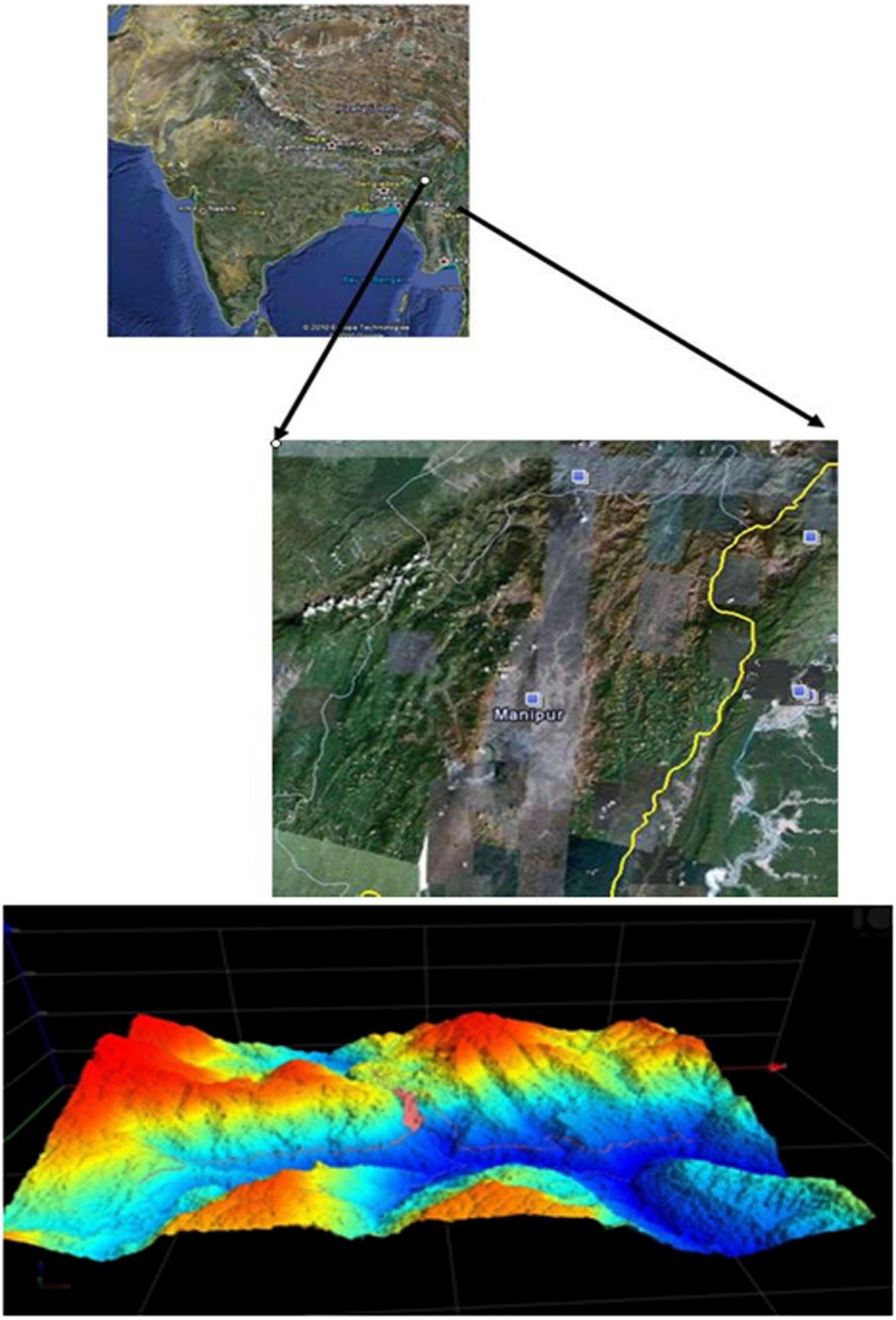

Figure 2: Location showing Nung Dolan Landslide & DME of Nung Dolan Landslide

Figure 3: The sliding area is not a small portion but larger areas are affected.

Methodology

The methodology involves geotechnical investigations of rocks as well as soils of the slide and slope stability studies of the area through kinematic studies in a systematic way in order to account for all the parameters responsible for instability. The investigations have been carried out through both laboratory and field studies. From the same, in addition to landslide vulnerability, the causative factors were also brought out, and there from mitigation strategies were evolved.

Results and Discussion

In the present study, the rock specimens which are prepared in such a way that both directions parallel as well as perpendicular to bedding planes are tested. The compression test samples with length: diameter ratio of 2:1 is employed for uniaxial compressive strength and samples with length: diameter ratio of 1:1 is used for tensile strength (Brazilian test).

Mean Compressive Strength is obtained from the tests (at least three samples) for the specimens parallel and perpendicular to bedding planes. Similarly, Mean Tensile Strength is also determined. It is observed from the results of compressive strength that samples perpendicular to the beddings give high strength value than those parallel to the beddings. It may be due to the presence of some structurally weak planar surfaces or some irregularity in the rocks that are not visible to the naked eye.

Table 1: Mean Compressive Strength of rock samples

Rock Specimens	Load (P) in kN	Mean Diameter (cm)	Radius (cm)	Compressive Strength $C = P/A(P/\pi r^2)$ in MPa
Parallel	55.04	3.29	1.65	64.75
Perpendicular	77.00	3.31	1.66	89.75
Mean	66.02	3.30	1.65	77.25

Table 2: Mean Tensile Strength of rock samples

Rock Specimens	Load P, kN	Mean Diameter, D(cm)	Mean Length, L (cm)	Tensile strength, $T = 2P/\pi DL$ (MPa)
Parallel	11.63	3.32	3.44	6.50
Perpendicular	13.96	3.32	3.43	8.00
Mean	12.80	3.32	3.44	7.25

Failure Envelope

The results obtained from these tests can be plotted using Mohr's stress circle in which the value of compressive strength are laid out to the right of zero on the horizontal axis while that of the tensile strength on the left of zero. In this method, the values of these strength parameters i.e. of the uniaxial compressive and tensile strengths are taken as diameters for their respective circles. A common tangent to both the circles is drawn as shown in the Fig.3. The point of intersection of this line with the Y-axis or τ-axis (Shear Stress) gives the value of cohesion, C and its slope represents the internal friction angle, ϕ. The point in which this tangent touches the right circle gives the value of normal stress, σ_n (along x-axis). By knowing these values, one can calculate the failure envelope, adopting the formula, $\tau = C + \sigma_n \tan\phi$.

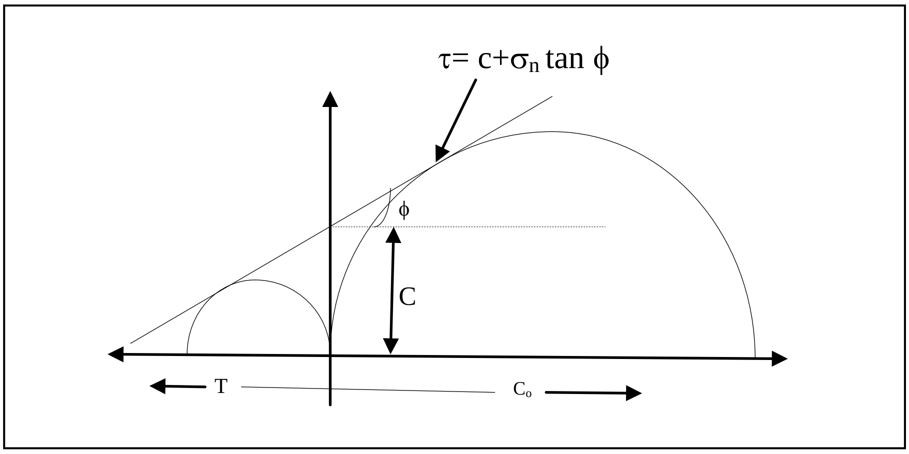

Figure 4: Mohr's failure envelope

Table 3: Strength parameters for samples perpendicular to the bedding

Rock sample	Cohesion, C (Mpa)	Internal friction angle, ϕ	Normal stress σ_n in (Mpa)	Shear strength, $\tau = C + \sigma_n \tan\phi$
A	14	60	7	26.124MPa
B	11.5	54	7	21.135MPa
C	12	55	7	21.997MPa
D	13	54	7,5	23.323MPa

Table 4: Strength parameters for samples parallel to the bedding

Rock sample	Cohesion, C in (Mpa)	Internal friction angle, ϕ	Normal stress σ_n in (Mpa)	Shear strength, $\tau = C + \sigma_n \tan\phi$
A	11	49	7.5	19.628 Mpa
B	12	52	7	20.959Mpa
C	7	56.5	3.5	12.288Mpa
D	6	58	7.5	18.003MPa

The Surma group of rock of the study area is classified on the basis of uniaxial compressive strength (UCS) and tensile strength based on Geological Society of London and Franklin & Brock, 1972 respectively and are shown in following Tables 5 and 6.

Table 5: Classification of Surma sandstones on the basis of UCS

Sample Nos.	Uniaxial compressive strength range (M Pa)	Description
Sample A	105-61	Very strong to strong
Sample B	74-71	Strong
Sample C	74-45	Strong to moderately strong
Sample D	106- 82	Very strong to strong

Table 6: Classification of Surma sandstones on the basis of tensile strength

Sample Nos.	Tensile strength range (MPa)	Description
Sample A	8-7	Moderate strength
Sample B	8-7	Moderate strength
Sample C	8-4	Moderate to low strength
Sample D	8-8	Moderate strength

Hence, the average UCS for the rock available in the present investigation is taken as 77.25 M Pa for slope stability analysis of RMR and SMR.

Subsequent to the geotechnical studies of rocks, SMR (Slope mass rating) and RMR (Rock mass rating) techniques were also used.

Rock and Slope Mass Rating

Field observations and measurements of discontinuities are the main method for finding the SMR. The present study is based on the slope mass rating (SMR) technique after Romana (1985) involving various parameters like orientations and conditions of discontinuities, attitudes of slope, degree of weathering, strength of rock mass and groundwater conditions.

The SMR technique is basically evolved from RMR system developed by Bieniawski (1989) which is based on the following six parameters: **i)** Uniaxial Compressive Strength of rock material, **ii)** RQD, **iii)** spacing of discontinuities, **iv)** condition of discontinuities, and **v)** water inflow through discontinuities and/ or pore pressure ratio, **vi)** Discontinuity orientation.

The "Slope Mass Rating" (SMR) is obtained from Bieniawski's RMR by adding a factorial adjustment factor; depending on the relative orientation of joints and slopes and another adjustment factor depending on the method of rock slope excavation.

$SMR=RMR + (F_1 \times F_2 \times F_3) + F_4$

The adjustment rating for joints is the product of three factors, as mentioned hereinafter:

i) F_1 depends on parallelism between joints and slope face strike, ranging from 1.00 to 0.15.These values match the relationship of $F_1= (1- \sin A)^2$ where A denotes the angle between the strikes of slope face and joints, with its absolute value.

ii) F_2 refers to joint dip angle in the planar mode of failure, ranging from 1.00 to 0.15 and matches the relationship of $F_2= \tan^2 B_j$ where B_j denotes the joint dip angle. For toppling mode of failure F_2 value remains 1.00.

iii) F_3 reflects the relationship between the slope face and joints dip angles, ranging from 0^0 to 60^0.

iv) F_4 refers to the method of excavation of the slope, which ranges from +15 (natural slope) to -8 (deficient blasting).

Slope characteristics and strength parameters of the landslide from the present study area is shown in Table 7.

Table 7: Slope characteristics and Strength Parameters of the Landslide area.

Station	Location	Attitude of Slope	Type of Rock	Attitude of Discontinuities (˚)	Degree of Weathering	Strength Qc (MPa)
Nung Dolan	24º46´8.2˝N 93º18´51.3˝E	65º/081º	Massive to thickly bedded sandstone.	66/057 80/163	Low	77.25

Table 8: Orientation of Discontinuities and Slopes

Station	Aj	Bj	as	Bs	Aj-as	Bj-Bs-180	Probable Failure
Nung Dolan	98	81	81	65	17	34	Wedge

Aj = joint dip direction; Bj = joint dip; as = slope direction; Bs = slope angle.

Table 9: Determination of RMR

Station	Strength	RQD	Spacing	jL	jR	jA	jC	Groundwater	RMR
Nung Dolan	7	17	15	2	2	1	4	15	58

jL=joint continuity or length; jR=joint roughness; jA= joint alteration; jC= joint condition factor.

Table 10: Calculation of SMR and stability classes

Station	Rating for adjustment factor				SMR	Class	Description	Stability	Failure	Support
	F_1	F_2	F_3	F_4						
Nung Dolan	4	0.1	-5	0	57.1	III	Normal	Partially unstable	Some joints or many wedges	Systematic

Kinematic Analysis of the Discontinuities

Hoek and Bray (2005), have established four modes of failures viz. Planar, Wedge, Circular or Rotational and Toppling. Modes of slope failures in jointed rock masses were examined kinematically using stereographic projection technique (Panet, 1969), which is purely geometric in nature. This technique was followed by Markland (1972), Goodman (1976), Hocking (1976), Cruden (1978), Lucas (1980), Hoek and Bray (1981), Matherson (1988) and Yoon et al.,(2002). The angular relationships between discontinuities and slope surfaces are applied to determine the potential and modes of failures (Kliche, 1999). Planar failure occurs when the strike trend of slope face and discontinuity plane are more or less parallel (±20°); dip angle of discontinuity plane is lower than the slope face, i.e, it must 'daylight' in the slope face and a release surface must be present in the rock mass to define the lateral boundaries of of the slide (Hoek and Bray,2005). According to Markland (1972) wedge failure occurs when the plunge of the intersection lines of discontinuity is less than the dip angle of the slope face and hence, these intersections must 'daylight' in the slope face. If the slope face and discontinuity planes are dipping in opposite direction, toppling failure may occur. Even if they are dipping in the same direction and the dip angle of the discontinuity plane is greater than the slope angle, there is probability of toppling.

Discontinuities data collected from the field are systematically processed and tabulated so that it can be effectively used to analyse stereographically using Rockpack III software, to determine the probable mode of slope failures (Figure 5).

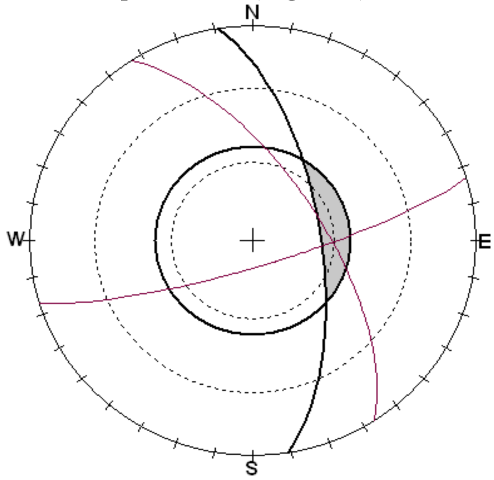

Figure 5: Stereo plot showing probable mode of failure.

It has been inferred that wedge failure is the probable mode of failure of the landslide where the intersection of joints (J1 & J3) lies in the shadow area by using Rockpack III, the plunge of the intersection lines of discontinuity is 60°/092°. Wedge failure is the mode of failure when the plunge of intersection lines of discontinuity is less than the dip angle of the slope face (Markland test).

In slope stability analysis, study of rock alone may not satisfy assessment of slope stability analysis. So it is required to assess certain properties of soil that can determine its engineering properties which is termed as Index properties. They are:

i) Water/moisture content.

ii) Bulk density.

iii) Specific gravity.

iv) Particle size distribution.

v) Consistency limits

Table 11: Results of Nung Dolan Soil testing

Moisture content(%)	Specfic gravity	Plastic Limit(%)	Liquid Limit(%)	Shrinkage Limit(%)	Plasticity index	Liquidity index	Consistency Index
12.91	2.60	22.02	26.50	30.22	4.48	-1.18	3.03

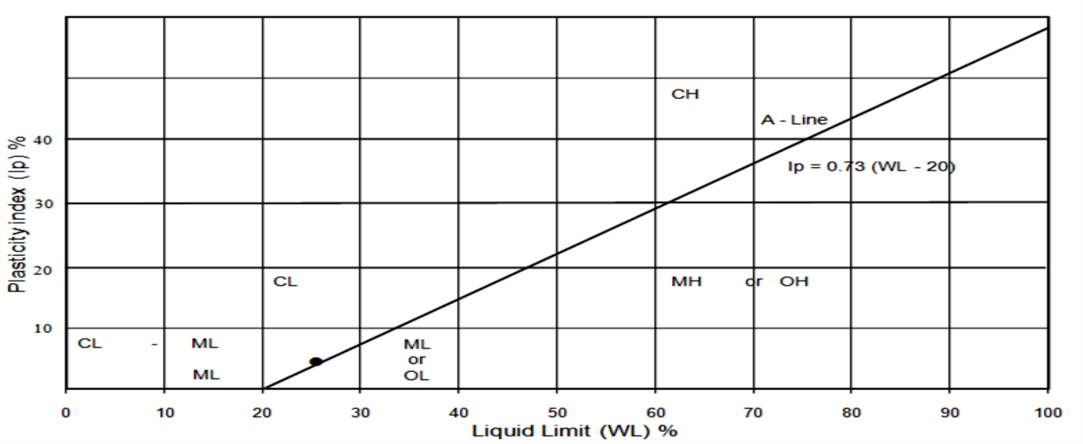

Figure 6: Plasticity chart of Soils

Table 12: Values of Liquidity index (I_L) and Consistency index (I_C) to consistency of soil (Murthy, 2007)

Consistency	Liquidity Index I_L	Consistency Index I_C
Semisolid or solid state	Negative	>1
Very stiff state	0	1
Very soft state	1	0
Liquid state (When disturbed)	>1	Negative

In the present study of soil properties of the slide, soil has less moisture content and from the plasticity index charts the soil samples come under CL group suggesting slightly plastic soil of inorganic nature. Hence we can conclude that the (-ve) value of **(-1.18)** liquidity index and the (+ve) value of **(3.03)** consistency index, indicates that the slope forming materials remains in solid state or semi solid state.

Causative Factors Responsible For Slope Failure

Detailed studies of the landslide have indicated that both natural and anthropogenic causes are main causative factors for the slope failures.

Natural Causes

i) Gravity: Gravity works more effectively on steeper slopes, but more gradual slopes may also be vulnerable.

ii) Geological factors: Geological structure and neotectonic activity have an inherent input in the area under investigation. High relief and steep slope prevailing in the area are not conducive for the stability of slope.

iii) Heavy and prolonged rainfall: Water is commonly the primary factor triggering a landslide. The region receives highest rainfall in the state. Therefore heavy and prolonged rainfall has led to oversaturation, causing loss in shearing resistance with the increment in weight. Surface runoff induces rapid underground circulation of water responsible for the building up of pore-water pressure, leading eventually to landslides.

Anthropogenic Causes

i) Cutting & deep excavation on slopes for roads, canals, construction of buildings, embankments and inappropriate disposal of debris after excavations.

ii) Blasting for road-cutting creates oscillations of a different frequency in the rocks, in the underground drainage system and in the shear strength of the slope materials in the area. These developments affect the forces operating on the slopes. Improper land-use and land-cover practices are the excessive hill slope cutting either due to construction of new road and widening of the existing roads has led to instability of slope.

iii) Blocking of surface drainage, loading of critical slopes and withdrawal to toe support promoting vulnerability of critical slopes.

Mitigative Measures

The mitigation measures involve the stabilization of the slide. Complete prevention of landslide is a very difficult task. However, the effects of landslides, especially the smaller ones and those induced by human activities, can be minimized. Because the vulnerability to landslide hazard is a function of location, type of human activity, and land use. So the following important Mitigative measures are suggested for the same.

i) Modification of slope geometry
  (a) Removing material from the area driving the landslide.
  (b) Reducing general slope angle.
ii) Drainage
  (a) Surface drains to divert water from flowing on to slide area.
  (b) Development of good and effective drainage system so that slope materials do not become oversaturated and trigger the slide.
iii) Retaining structure

(a) Rock fall attenuation or stopping system (rock trap ditches, benches, fences and wall.
(b) Retention net for rock slope faces.
(c) Cast-in-situ reinforced concrete walls

iv) Internal slope reinforcement
(a) Rock bolts
(b) Micro piles
(c) Stone or lime/cement columns

Conclusions

Depending upon the nature and suitability of the prevailing conditions different techniques have been applied to study the landslides by various workers. In the present study, geotechnical investigations of rocks as well as soils, SMR technique and Kinematic analysis of the discontinuities of rocks using stereographic projection techniques have been employed. Test results of compressive strength of rock samples made perpendicular to bedding have higher strength values in comparison with those made parallel to the beddings which may be due to the presence of some structurally weak planes or surfaces or some irregularity in the rocks that are not visible to the naked eye. The structural features in rocks and anthropogenic factors like faulty design of the road cut and improper method of blasting might be the main causes of the rock slide/fall.The present study leads to the conclusion that rapid pace of anthropogenic interferences on the natural slope and prevailing unfavourable geological discontinuities along with rain water action have played an important role in triggering the rock fall.

Failure modes evaluated using SMR and Kinematic Analysis Methods are more or less in conformity for the slide. RMR value is 58 of fair category belonging to class III and SMR value is 57.1, falling under partially unstable condition, suggesting systematic support, viz: systematic bolting, dental treatment, net and toe drains. The results obtained from kinematic analysis and field observations are in conformity with the probable mode of failure. Kinematic analysis based on the discontinuity data suggests wedge type of failure of the landslide.

Acknowledgements

The grant by the Department of Science & Technology, Government of India, New Delhi and the initiative of Dr Bhoop Singh, Director/Scientist ‘F’, DST, New Delhi, is gratefully acknowledged. My sincere thanks are due to Department of Earth Sciences, Manipur University for the Laboratory facilities and for fruitful discussions in the present study.

References

- Bieniawski Z.T. 1989, Engineering Rock Mass Classifications. Wiley, New York.
- Cruden, D.M., 1978, Discussion of Hocking’s paper “A method for distinguishing between single and double plane sliding of tetrahedral wedges”. Int.Jour. Rock Mech. Min. Sci. Geomech. Abstr. 15, 217.
- Goodman, R.E., 1976, Methods of Geological Engineering in Discontinuous Rocks. West Publishing, San Francisco.
- Hocking, G., 1976, A method for distinguishing between single and double plane sliding of tetrahedral wedges. Int.Jour. Rock Mech. Min. Sci. Geomech. Abstr. 13.
- Hoek, E. and Bray, J.W., 1981, Rock Slope Engineering. 3rd Edn. London: Institution of Mining and Metallurgy, 57, 92, 96, 150.
- Ibotombi, S.,1998, On the Geology of Manipur. In- Sov., 9th Manipur Science Cong., 12-18
- Kliche, C.A., 1999, Rock Slope Stability SME, Littleton, CO.
- Lucas, J.M., 1980, A general stereographic method for determining possible mode of failure of any tetrahedral rock wedge. Int. Jour. Rock Mech. Min. Sci. Geomech. Abstr., 17, 57 – 61.

- 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. Coll. Rock Mech. Res., Rep., 19, 10.
- Matherson, G.D., 1988, The collection and use of field discontinuity data in rock slope design. Quart. Jour. Engg. Geology, 22, 19 – 30.
- Murthy, V.N.S., 2007: Soil Mechanics and Foundation Engineering, Geotechnical
- Engineering Series, Satish Kumar Jainfor CBS Publisher & Distributor, New Delhi, first
- edition, pp 81-119.
- Panet, M., 1969,Discussion on “Graphical stability analysis of slopes in jointed rock”. By K.W. John. Jour. Soil Mech. Found. Div., Proc. ASCE. 95 (SM2) 685 – 686.
- Yoon, W.S., Jeong, U., Choi, J.W., Kim, J.H., Kim, W.Y. and Kim, C.S., 2002, Slope failure index system based on the behaviour characteristics, SFi – System. Jour.of the Korean Geotech. Soc., 18, 23-37.