TECHNICAL
FINAL REPORT

LANDSLIDE HAZARD ZONATION BETWEEN NONEY-NUNGBA ALONG NH-53 AND GEOTECHNICAL INVESTIGATION FOR TWO SLIDES

Submitted To:

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PROJECT PROFILE

Project Tittle	:	Landslide Hazard Zonation between Noney-Nungba along NH-53 and Geotechnical Investigation for two slides
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1. INTRODUCTION

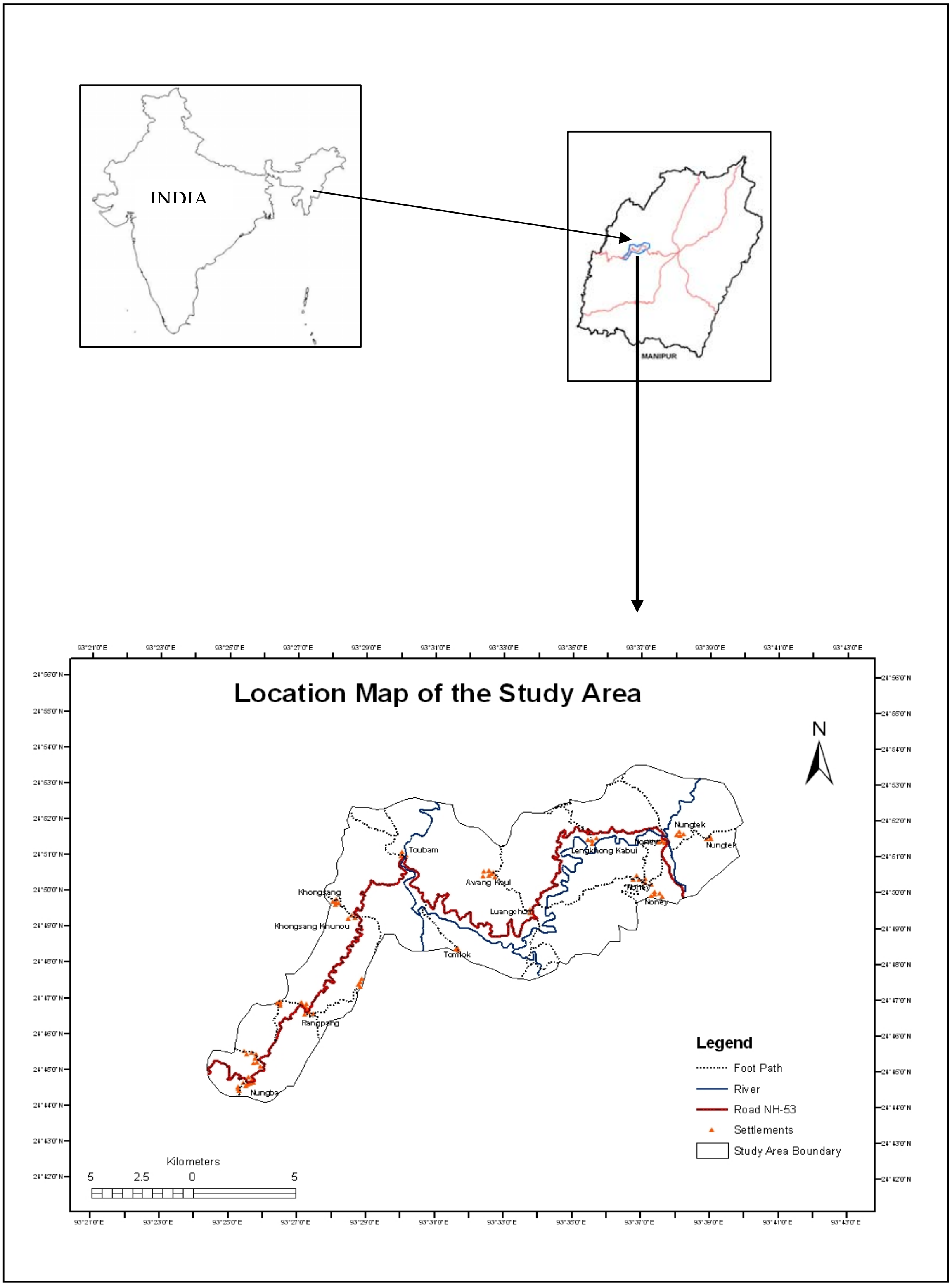
Landslides are among the major hydro-geological hazards that affect large parts of the country. Most of the north-eastern region is bristling with landslides of bewildering variety. Northeast region, because of its continued evolution, fragile geological formation and structures, is highly prone to mass movement causing landslides. Since landslides are mostly triggered by events of heavy rainfall and seismicity, which could be followed by flood in the plains, the local populace fills the impact of this location caused by landslides.

Manipur is a small hilly state of northeast India which is situated between 23°50' N to 25°42' N Latitude and 93° 00' E to 94° 45' E Longitudes and occupies an geographical area of about 22,327 sq km and connected with adjoining states by three national highways namely NH-39, NH-53 and NH-150 which are the lifelines of the state. The present study had been carried out along a part of **NH 53** between **Noney–Nungba** about **63km** linear stretch. The project area comprises an area of about **173 sq.km.** with geographical coordinate of **24°44′02″N to 24°53′35″N** and **93°24′ 17″ E to 93°40′ 01″ E .** An area of 173sq.km of Barak catchment has been taken to study the various parameters for landslides hazard zonation affecting the NH-53 between Noney-Nungba. During the tenure of the project 18 incidence of landslides have been identified. Most of the landslides fall into active and old slides. The major causes of the slides are both natural and anthropogenic. Geological formation along the road section between Noney-Nungba along NH-53, belong to Barail Group and Surma Group. The constituent litho units are susceptible to weathering and erosion leading to slopes failure and mast wasting on moderate to large scale. The stratified nature of rocks, affected by deformation, plays important role in causing landslides. The geotechnical aspects of the soil and rock types will be investigated to determine their shear strength and other lithological, structural, geomorphological as well as hydrogeological properties will also be studied in the present work.

1.1 OBJECTIVES

1. To study geomorphology, geology and structural parameters for slope stability.
2. To study hydrological conditions.
3. To study soil and rock mechanical properties.
4. To prepare a detailed map from the sites specific studies, risk assessment and to develop preventive measures.

LOCATION MAP OF THE STUDY AREA



1.2 GEOLOGY OF THE STUDY AREA

Geological formation occurring in the study area belongs to the Barail and the Surma groups which are represented by shale, siltstones, sandstones, conglomerate and recent alluvium. These rock formations are tectonically deformed and highly weathered. A lithtectonic map of the study area has been developed on the basis of visual interpretation of LISS III imagery, available geological map on regional scale and field survey. Major part of the study area is occupied by the Barail Group of rocks. A simplified stratigraphic succession of the study area is given in the table 1. (*Modified after Ibotombi, 1998, & Okendro*)

Table 1 Stratigraphic Succession of the Study Area

Litho-units and age	**Description of rocks**
The Surmas (Upper Oligocene to Miocene)	Shale, sandy-shale, siltstone, ferruginous sandstone, massive sandstone. Alternations of sandstone and shale with minor conglomerate. Transitional character from flysch to molasses sediments.
~~	~~~~~~~~~~~~~~ ***Stratigraphic Break***~~~~~~~~~~~~~~~~~
The Barails (Upper Eocene to Oligocene)	Massive to thick bedded sandstone. Alteration of shale and sandstone with carbonaceous matter. Intercalation of bedded sandstone with shales. Flysch of turbidite character.

The area between Noney and Awangkhul is covered with highly deformed and crushed material and recent alluvium. The Aleng River near Noney is following an anticlinal axis of eroded hinge of fold. However, towards Awangkhul thickness and frequency of sandstone beds and degree of deformation increase indicated by the presence of minor folds, faults and variation in the altitude of beds.

After Awangkhul, rock formations are represented by the rhythmic intercalations of thinly laminated shales, sandy shale, thickly bedded siltstones and sandstones. Thickness of sandstone beds increases towards the top. Rocks are highly folded as overtuned fold. Some parts of the area are covered with thick alluvium and luxuriant bamboo growth, despite this, the area is vulnerable to landslides.

The Irang River is running along a faulted anticlinal axis. The rock formations between Awangkhul and Taobam are represented by the intercalations of thinly bedded sandstones, siltstones and light earthy coloured shales, which is highly deformed due to secondary generation of folding. The rock units of the area between Taobam (The Irang River) and Khongsang are highly deformed as manifested in folds, faults and are fractured and jointed. The intercalations of shales, siltstones and sandstones gradually become thicker towards

Khongsang village. Near Khongsang, the rocks are moderately to intensely weathered with thick cover of recent alluvium. Due to folding and faulting, at places, rock formations have almost become vertical. A number of minor folds and different sets of joints are present between Khongsang and Rengpang.

Rock formation between Rengpang and Nungba are represented by thickly bedded to massive sandstones with sandy-shales and silty-shales intercalations. However, near Nungba the thickness and frequency of sandstone beds again decrease. At a distance of about two kilometer north of Nungba, there is unconformable contact between the Barail and the Surma groups, represented by a thin band of metamorphosed conglomerate**.** Presence of thin bands of coal, light grey to brownish coloured sandstone and reddish coloured shales indicate that during the deposition of Surma the basin become very shallow.

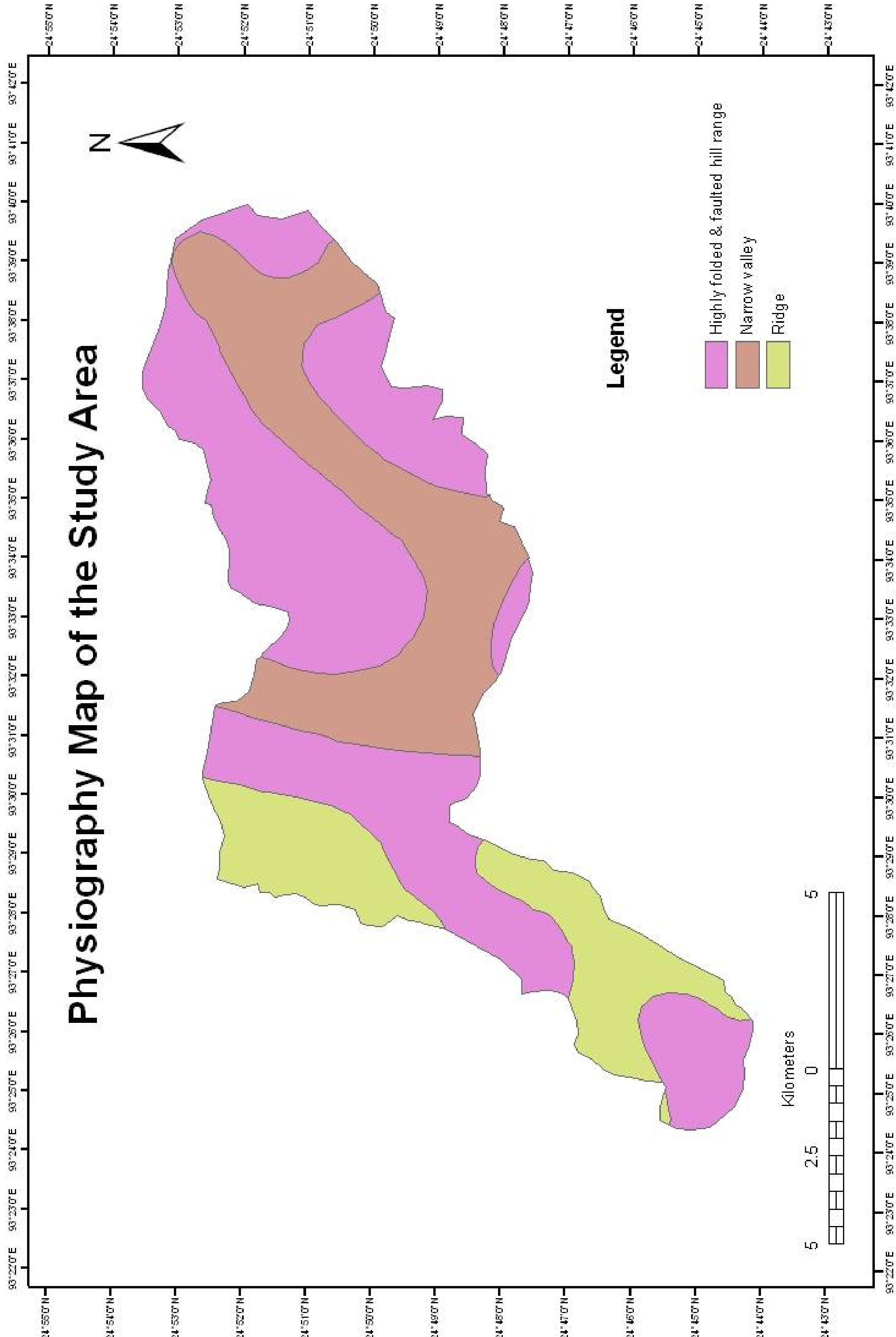
1.3 PHYSIOGRAPHY OF THE AREA

The entire study area is the hilly terrain. These hill ranges are parallel to sub-parallel, trending NNE-SSW and N-S directions. General height of these hill ranges varies between 200m to 1520m from mean sea level. The entire area is drained by rivers namely Ijai (Iyai), Leimatak or Apen or Pen river and Irang or Toubam river and their tributaries. The area under investigation can be broadly divided into following physiographic units:

Highly folded and faulted hills ranges: - Highly folded and faulted hills cover major part of the area. They are arranged in parallel to sub-parallel and sometimes rectilinear in nature. This is either due to folding and /or faulting.

Narrow Valley: - In the study area valleys are occupied by the major river like the Ijai, the Leimatak, the Irang, the Aleng rivers and their tributaries. General trends of the major valley are NNW-SSE and NE-SW. These rivers are flowing either along the faulted zones or axes of folds.

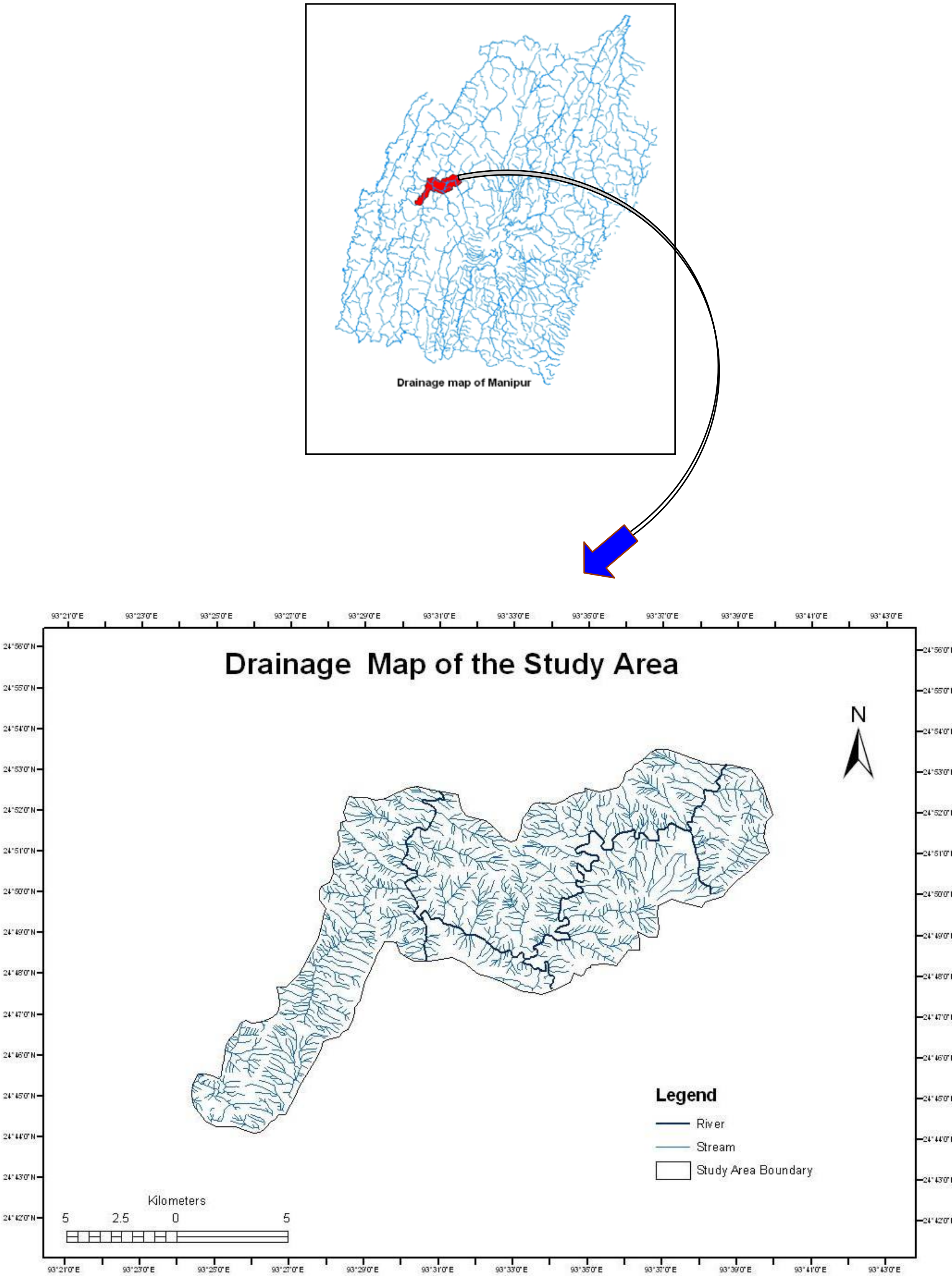
Ridges: - Ridge occupied the smaller portion of the NW & SW parts of the study area. The phisiography map of the study area is shown in the map below (*source: Physiography map of Manipur).*


Physiography Map of the Study Area
N
Legend
Highly folded & faulted hill range
Narrow valley
Ridge
Kilometers
5 2.5 0 5

1.4 DRAINAGE SYSTEM

The drainage system of an area plays an important role in the hydrogeological condition. According to Thornbury, (1954) it defined as the particular plan or design, which the individual stream courses collectively form. The two most important river basins of the state Manipur are Manipur river basin and Barak river basin. The Manipur river basin drains all the eastern half of the state including the central plain through Chindwin river into the Irrawadi drainage system of Myanmar whereas western half of the state is drained by Barak river basin through Dhansiri river into the Brahmaputra – Ganga drainage system.

The drainage network of the study area falls into the Barak river basin. As mentioned above, Barak river, the largest river of Manipur drains the western half of the state having about 250 sq. km. of its river valley area. It rises from northern ranges of the state about 16 km east of Mao and follows a south westerly course. The most important tributaries of Barak river are Jiri, Tipai, Juko, Makru, Irang, Leimatak, Maklang etc. The Ijai or Iyai, Leimatak and Toubam or Irang river are main tributaries fall into the study area. The drainage map of the study area is prepared from the 1:50,000 scale toposheet of Survey of India (SOI) shown in the fig. below. Various types of drainage pattern are identified by analysis the different stream arrangement found in the study area. Sub-trellis type of drainage pattern are found in the Leimatak and the Irang river near Awangkhul. It indicates that the area might be underlain by folded sedimentary sequence of varying resistance. Radial pattern in which streams originating from a common higher point or peak diverge in all directions are found in Rengpang area and rectangular drainage pattern is observed in the course of Ijai river takes several at right angle turns between Haochong and Toubam villages. In the study area sudden widening followed by compressed valley can be seen along the course of the Ijai river near Langkhong Kabui village, may be due to the sudden appearance of hard and compact lithology or some structural complexities in the area showing anomalous drainage pattern.


Drainage map of Manipur
Drainage Map of the Study Area
N
Legend
River
Stream
Study Area Boundary
Kilometers
5 2.5 0 5

2 LANDSLIDES

A Landslide is the downward and outward movement of slope forming materials, composed of rock, soil, artificial fills (dumping) or a combination of all these along the surface of separation by falling, flowing under a fast or slow rate, but under the action of gravitational force and where the triggering factor may be natural or anthropogenic. Landslide is a major geological hazard, which poses serious threat to human population and various other infrastructures like highways, rail route and civil structures like dams, buildings and structures. Expansion of urban and recreational developments on hill areas result in ever increasing number of residential and commercial properties that are often threatened by landslides. Landslides also occur very often during other major natural disaster such as earthquakes, floods and volcanoes. Since the land route are often disturbed by landslides, they cause major hurdles in mobilizing relief and reconstruction efforts.

2.1 TYPES AND CLASSIFICATION

Table 2 shows a schematic landslide classification adopting the classification of Varnes 1978 and taking into account the modifications made by Cruden and Varnes, 1996. Some integration has been made by using the definitions of Hutchinson (1988) and Hungr et al 2001.

Table 2 Landslide classification (Varnes 1978)

<table>
<tr><th colspan="3" rowspan="3">Type of movement</th><th colspan="3">Type of material</th></tr>
<tr><th rowspan="2">Bedrock</th><th colspan="2">Engineering soils</th></tr>
<tr><th>Predominantly fine</th><th>Predominantly coarse</th></tr>
<tr><td colspan="3">Falls</td><td>Rock fall</td><td>Earth fall</td><td>Debris fall</td></tr>
<tr><td colspan="3">Topples</td><td>Rock topple</td><td>Earth topple</td><td>Debris topple</td></tr>
<tr><td rowspan="3">Slides</td><td>Rotational</td><td></td><td>Rock slump</td><td>Earth slump</td><td>Debris slump</td></tr>
<tr><td rowspan="2">Translational</td><td>Few units</td><td>Rock block slide</td><td>Earth block slide</td><td>Debris block slide</td></tr>
<tr><td>Many units</td><td>Rock slide</td><td>Earth slide</td><td>Debris slide</td></tr>
<tr><td colspan="3">Lateral spreads</td><td>Rock spread</td><td>Earth spread</td><td>Debris spread</td></tr>
<tr><td colspan="3" rowspan="3">Flows</td><td>Rock flow</td><td>Earth flow</td><td>Debris flow</td></tr>
<tr><td>Rock avalanche</td><td></td><td>Debris avalanche</td></tr>
<tr><td>(Deep creep)</td><td colspan="2">(Soil creep)</td></tr>
<tr><td colspan="3">Complex and compound</td><td colspan="3">Combination in time and/or space of two or more principal types of movement</td></tr>
</table>

2.2 CAUSES OF LANDSLIDES

The causes of landslides are usually related to instabilities in slopes. Causes may be considered to be factors that made the slope vulnerable to failure, that predispose the slope to becoming unstable. The trigger is the single event that finally initiated the landslide. Thus, causes combine to make a slope vulnerable to failure, and the trigger finally initiates the movement. Landslides can have many causes. Usually, it is relatively easy to determine the trigger after the landslide has occurred. Although it is generally very difficult to determine the exact nature of landslide triggers ahead of a movement event. Landslide causes can be broadly divided into two categories, i.e. natural and anthropogenic. The causative factors of landslide are shown in fig1.

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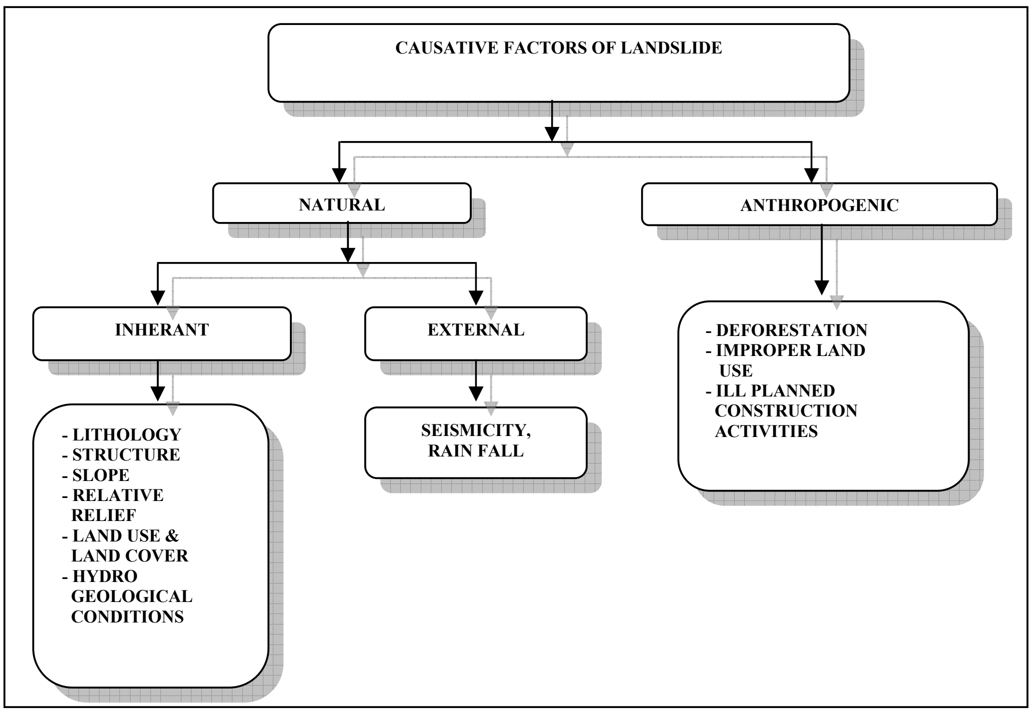


Fig 1: Causative factors of landslide

3. REVIEW OF THE PREVIOUS WORK ON LANDSLIDE HAZARD ZONATION

A number of workers have attempted landslide hazard zonation by using Remote Sensing and GIS techniques. Ali Yalcin and Fikri Bulut 2007; Lulseged Ayalew et.al., 2005, Carrara et.al., 1992; Mantovani et.al., 1996; Varnes 1984; Van Westen et.al. 1992, 1996; Crozier 1986 etc. attempted to review the underpinning issues, concepts, objectives and methodology for ultimately reducing hazard and risk arising from landslides. In Indian context many workers like Pachauri and Pant 1992; Gupta and Joshi 1990; Anbalagan 1992, Anbalagan and Singh 1996; Sharma 2008; Saha et.al., 2002; Nagarajan et.al., 1998, etc. have carried out the studies of landslides in different regions of the country. In Manipur, Department of Earth Sciences, Manipur University had also worked on landslides including hazard zonation, investigation, and mitigation measures etc. using different techniques.

4. PREPARATION OF LANDSLIDE HAZARD ZONATION MAP

A landslide hazard zonation is a division of the land surface into areas, and the relative ranking of these areas according to degrees of actual or potential hazard from landslides on slopes (Varnes, 1984). This is a method to evaluate the risk where there is the potential for landslides. It is an important tool for designers, field engineers and geologists, to classify the land surface into zones of varying degree of hazards based on the estimated significance of causative factors which influence the stability (Anbalagan, 1992). The landslide hazard zonation map, in short called LHZ map, is a rapid technique of hazard assessment of the land surface (Gupta and Anbalagan, 1995).

Detailed landslide hazard zonation maps (LHZ) will be prepared in areas susceptible to landslides causing heavy losses to life and property. Focus on landslide hazard zonation on the following scales:

- 1:50,000 - 1:25,000 for regional planning
- 1:15,000 -1,10,000 for district level planning
- 1:5,000-1:2,000 for site specific micro zonation (This includes the following aspects):
    - Preparation of engineering geologic profile of a landslide indicating slope angle, location of slip surface, nature of soils & rocks along the slip surface, surface and sub surface ground water conditions and weathering profile.
    - Current landuse practices and the vegetative cover.
    - Information on meteorological parameters such as; intensity, duration and frequency of significant rain events etc.

From the exhaustive literature survey and the field checks, following geo-environmental factors are found which are playing a significant role in causing slope instability problems in the area.

1. Slope Aspect
2. Slope Morphometry
3. Landuse/Landcover
4. Dip Slope Relation
5. Rockmass Strength
6. Drainage
7. Geology
8. Ridge/Crest Line
9. Road (Anthropogenic factor)
10. Tectonic/Lineament
11. Relative Relief

4.1 METHODOLOGY

The present study deals with landslide hazards which are occurring frequently and effecting severely in the study area. The methodology is based on the guidelines of the LHZ mapping (Ambalagan, 1992 and Bureau of Indian standard, BIS 1998). LHZ map of the present study area has been prepared on the basis of varying degree on the estimated significance of the causative factors of instability like lithology, soil type, structure, slope morphometry, relative relief, land use and land cover and hydrogeological condition.

A GIS software like ARC-GIS 9.0, ARC VIEW 3.2 and ERDAS Imagine 8.7 was used for integrating different thematic maps and assigning their combined effect. These thematic maps were quantified by giving them a relative score. Flow chart structure of methodology of Landslide Hazard Zonation, Geotechnical studies and Risk Assessment & Preventive Measures are shown in the fig. 2

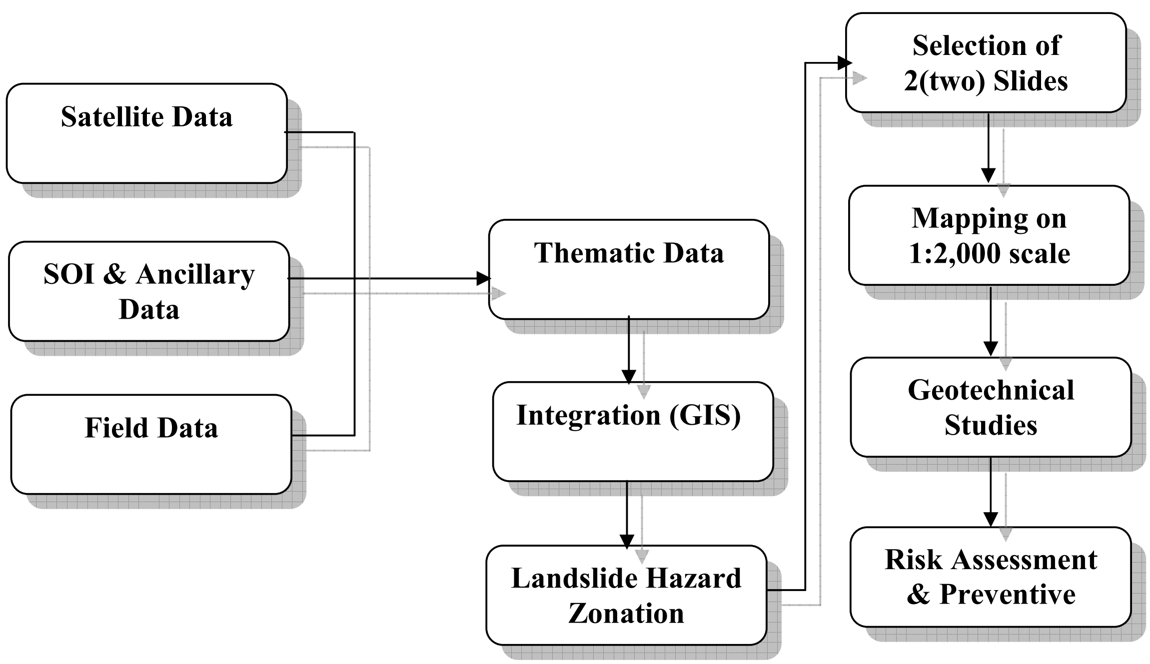


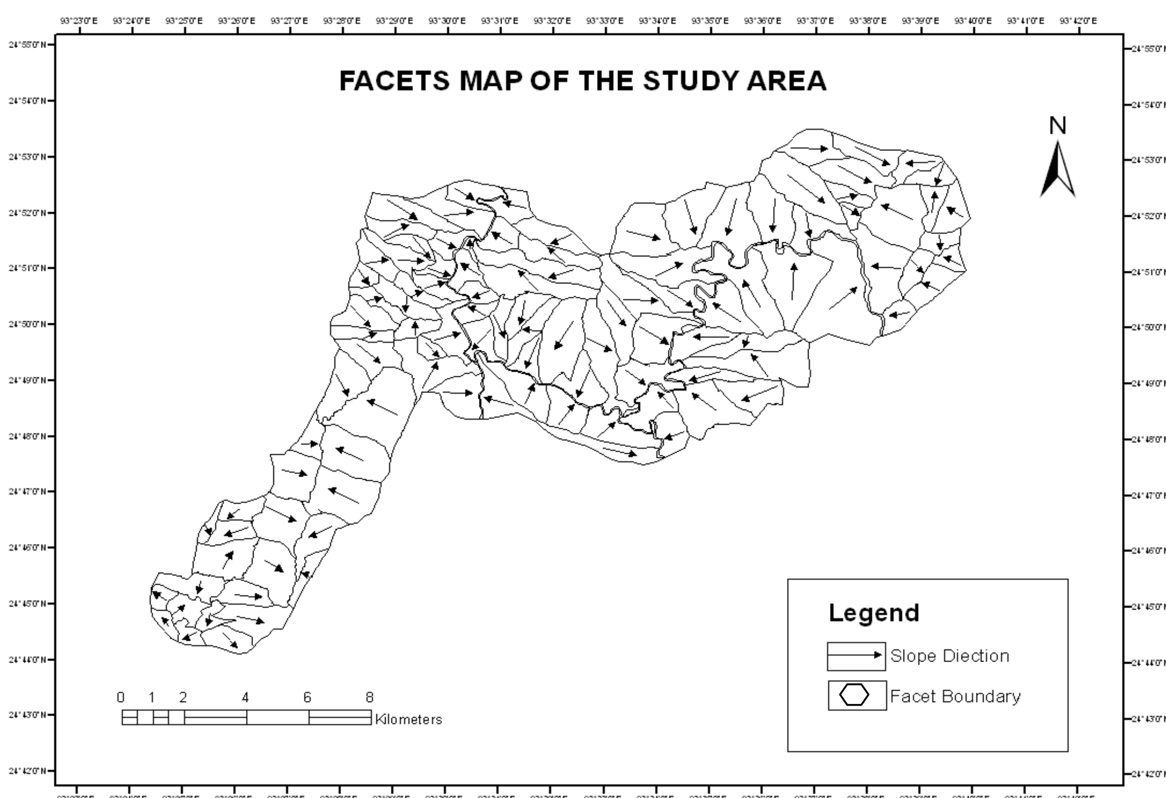
Fig. 2: Flow chart outlining the methodology of Landslide Hazard Zonation, Geotechnical studies and Risk Assessment & Preventive Measures map Generation

4.2 PREPARATION OF THEMATIC/BASE MAPS

The thematic/base maps of the study area were prepared using the Survey of India Toposheets (SOI), 1: 50,000 scale, Landsat Imageries, Soil map (Soil Survey of India) and available geological maps followed by detailed field survey. The base maps are used as a reference map for field survey, identification of landuse/landcover patterns, active and old landslides, anthropogenic activities and other related analysis.

4.3 PREPARATION OF FACET MAP

Facet is a polygonal area of mountainous terrain which has more or less similar characters of slope, showing consistent slope direction and inclination. The slope facets are generally delimited by ridges breaks in slope, streams, spurs, gullies and rivers etc. The facet maps form the basis for the preparation of thematic maps in general and LHZ mapping in particular and individual facet is the smallest mappable unit. In all 109 facets including sub facets have been delineated from the study area on the basis of visual interpretation of topographic maps.

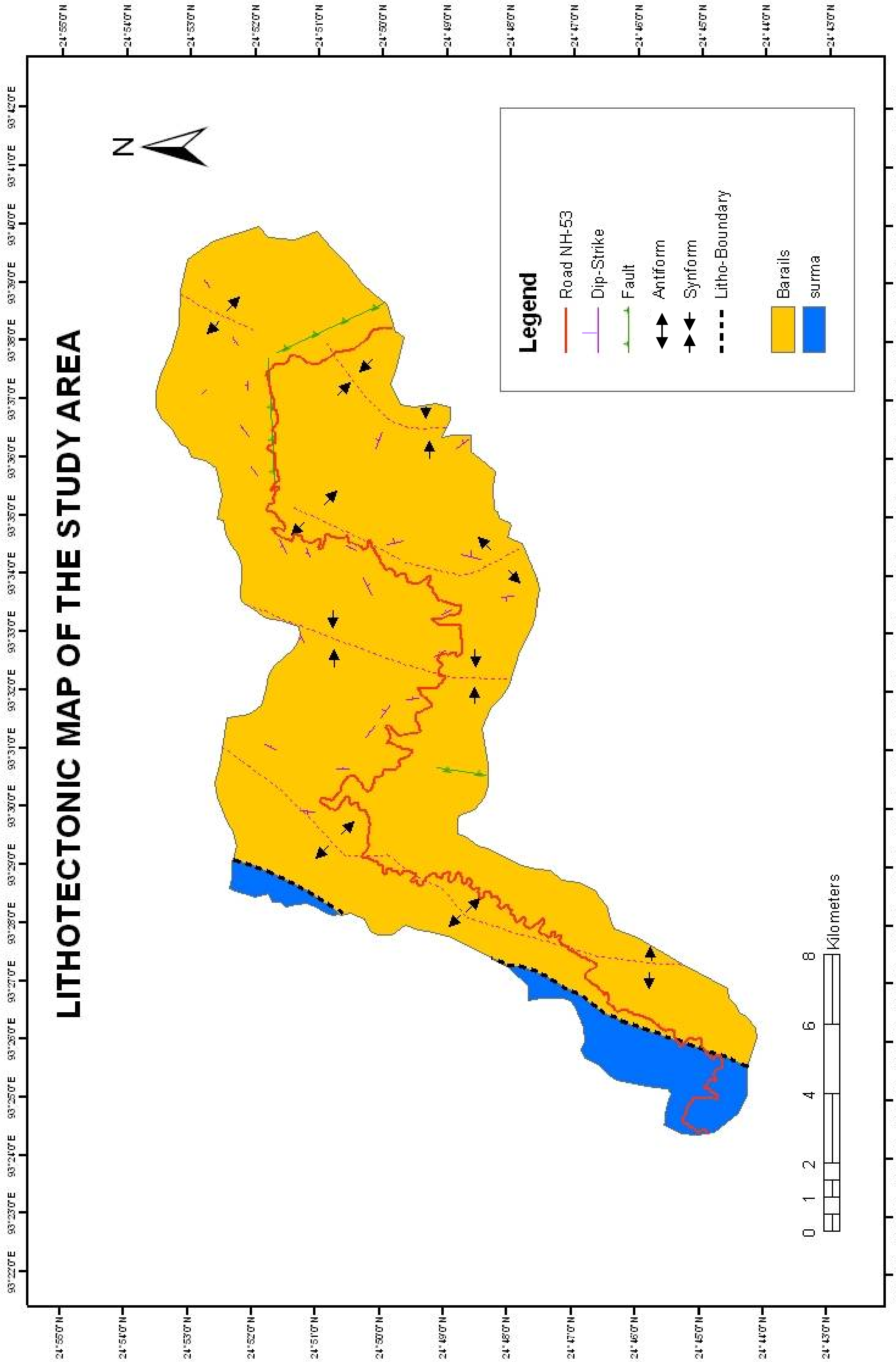


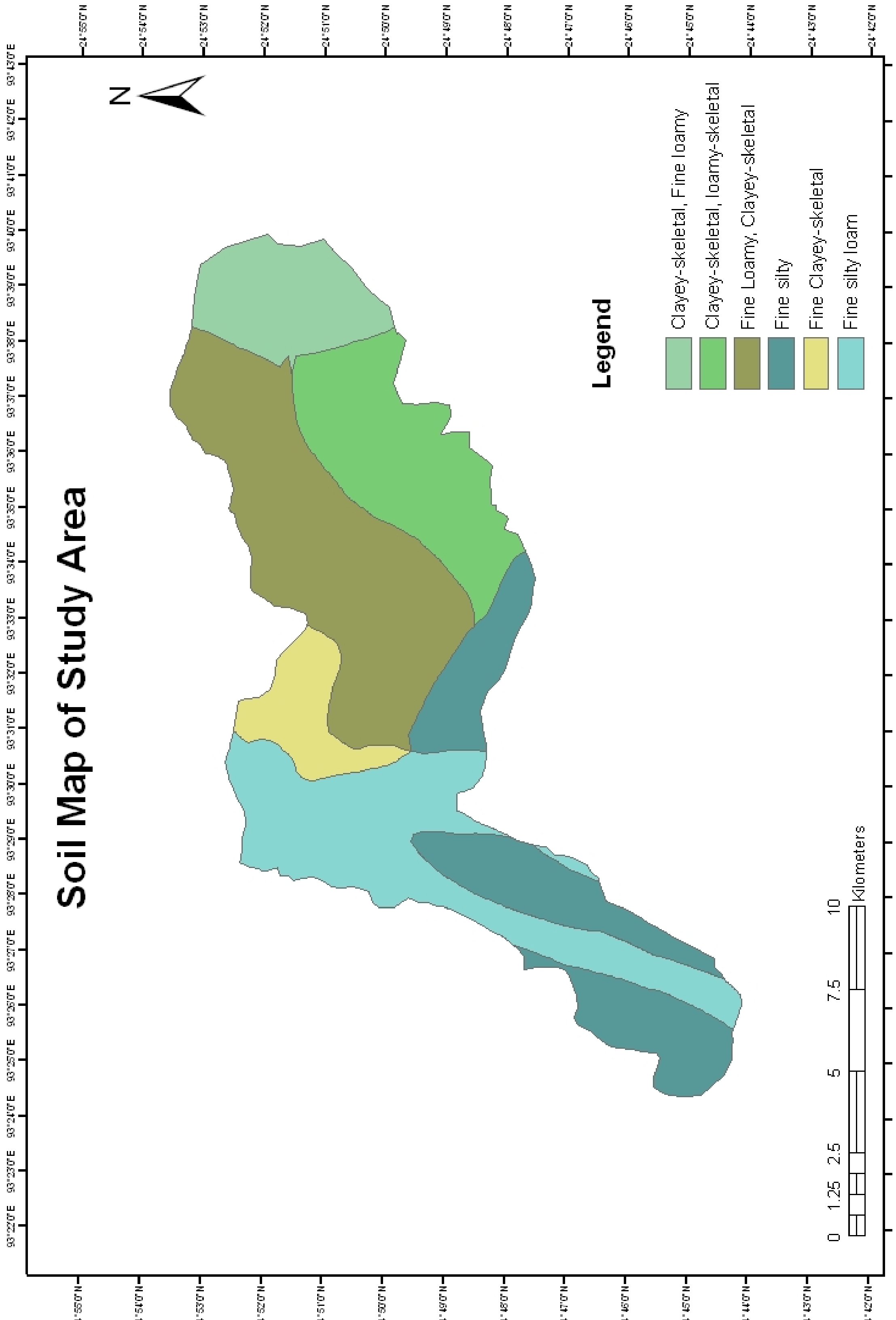
4.4 LITHOLOGY

The rocks in the study area are occupied by the Barail and Surma Groups of rocks. The Barail Group is characterized by alteration of sandstones and shale showing intercalations, sometimes thickly bedded sandstone beds showing typical turbidite character at places whereas Surma Group is characterized by shale, sandy-shale, siltstone, ferruginous sandstone, massive sandstone etc. They are sometimes characterized by alternations of sandstone and shale with minor conglomerate showing transitional character from flysch to molasses sediments. Rock types such as sandstone, shale, siltstone etc. are highly fractured and weathered in nature. So, they are relatively very weak and very common to easy for slides. These groups of rocks are almost covered with tertiary sediments. The slope forming materials predominantly consisting of debris made up of silty and sand mixed with some clay along with small amount of rock debris. The individual lithounits of the area are difficult to mapped in the 1:50,000 scale because of their thin and fragile intercalation nature of rock types. So they are treated here as above two groups of rocks. These lithotypes fall into the Type II of LHEF rating scheme (BIS, 1998) assigning with their relative rating values multiplied with the corresponding correction factor of weathering.

Table 3: Lithological Rating Scheme (after Anbalagan, 1992 & BIS, 1998)

Lithology	Type	Rating
Rock Type	Type I Quartzite and limestone Granite and gabbro Gneiss	 0.2 0.3 0.4
	Type II Well cemented sedimentary rocks dominantly sandstone with minor beds of claystone Poorly cemented sedimentary rocks dominantly sandstone with minor clay-beds	 1.0 1.3
	Type III Slate and phyllite Schist Shale inter-bedded with clayey and non clayey rocks Highly weathered shale, phyllite and schist	 1.2 1.3 1.8 2.0
Soil Type	Older well-cemented fluvial fill material Clayey soil with naturally form surfaces Sandy soil with naturally form surface (alluvial) Debris comprising mostly rock pieces mixed with clayey/sandy soil (colluvial) ---- Older well compacted ---- Younger loose material	0.8 1.0 1.4 1.2 2.0
Correction factor for weathering i) Highly weathered – rock discolored, joint open with weathered product, rock fabric alter to a large extent --- correction factor C_1 ii) Moderately weathered – rock discolored with fresh rock patches weathering more around joint planes but rock intact in nature ---Correction factor C_2 iii) Slightly weathered – rock slightly along joint planes, which may be moderately tight to open intact rock --- correction factor C_3. The correction for weathering to be multiplied with the fresh rock rating. For rock type – I, $C_1 = 4$, $C_2 = 3$, $C_3 = 2$ For rock type – II, $C_1 = 1.5$, $C_2 = 1.25$, $C_3 = 1$		


LITHOTECTONIC MAP OF THE STUDY AREA
N
Legend
Road NH-53
Dip-Strike
Fault
Antiform
Synform
Litho-Boundary
Barails
surma
0 1 2 4 6 8
Kilometers



Source: Soil map of Manipur, prepared by Soil Survey of India

4.5 STRUCTURE

Structures of the area include bedding planes, several set of joints, faults and folds etc. The structural discontinuity in relation to the direction and inclination of slope has greater influence on the stability of slope. The structural data have been superimposed on the lithological map and observed structural details are plotted on Georient 9.0 software and preferred orientation and possible failure mode (planar or wedge failure) is obtained for the facets occupied by bedrock. According to Anbalagan (1992) the following three types of relationships among these variables are categorised.

a) The extend of parallelism between the directions of the discontinuity or the line of intersection of two discontinuities and the slope.
b) Steepness of the dip of the discontinuity or plunge of the line of intersection of two discontinuities.
c) The difference in the dip of discontinuity or plunge of the line of intersection of two discontinuities to the inclination of slope.

If the plane of discontinuity or the line of intersection of two discontinuities tends to be parallel with the direction of inclination of slope face, the risk factor of slope failure increases. If the inclination of slope is more than the dip of discontinuity or plunge amount of line of intersection two discontinuity planes, the failure potential remains high.

The following Landslide Hazard Evaluation Factor (LHEF) rating scheme of structural conditions have been assigned for calculation of Total Estimated Hazard.

Table 4: Structural Rating Scheme (after Anbalagan, 1992 & BIS, 1998)

Relationship of structural discontinuity with slope		Category	Ratings
Relationship of parallelism between the slope and discontinuity	I	>30°	0.20
	II	21°-30°	0.25
	III	11°-20°	0.30
	IV	6°-10°	0.40
	V	<5°	0.50
Relationship of dip of discontinuity and inclination of slope.	I	>10°	0.3
	II	10°-0°	0.5
	III	0°	0.8
	IV	-10°- 0°	1.0
	V	>-10°	0.2
Dip of discontinuity	I	<15°	0.20
	II	16°-25°	0.25
	III	26°-35°	0.30
	IV	36°- 45°	0.40
	V	>45°	0.50
Depth of soil cover		<5 m	0.65
		6-10 m	0.85
		11-15 m	1.30
		16-20 m	2.0
		>20 m	1.20

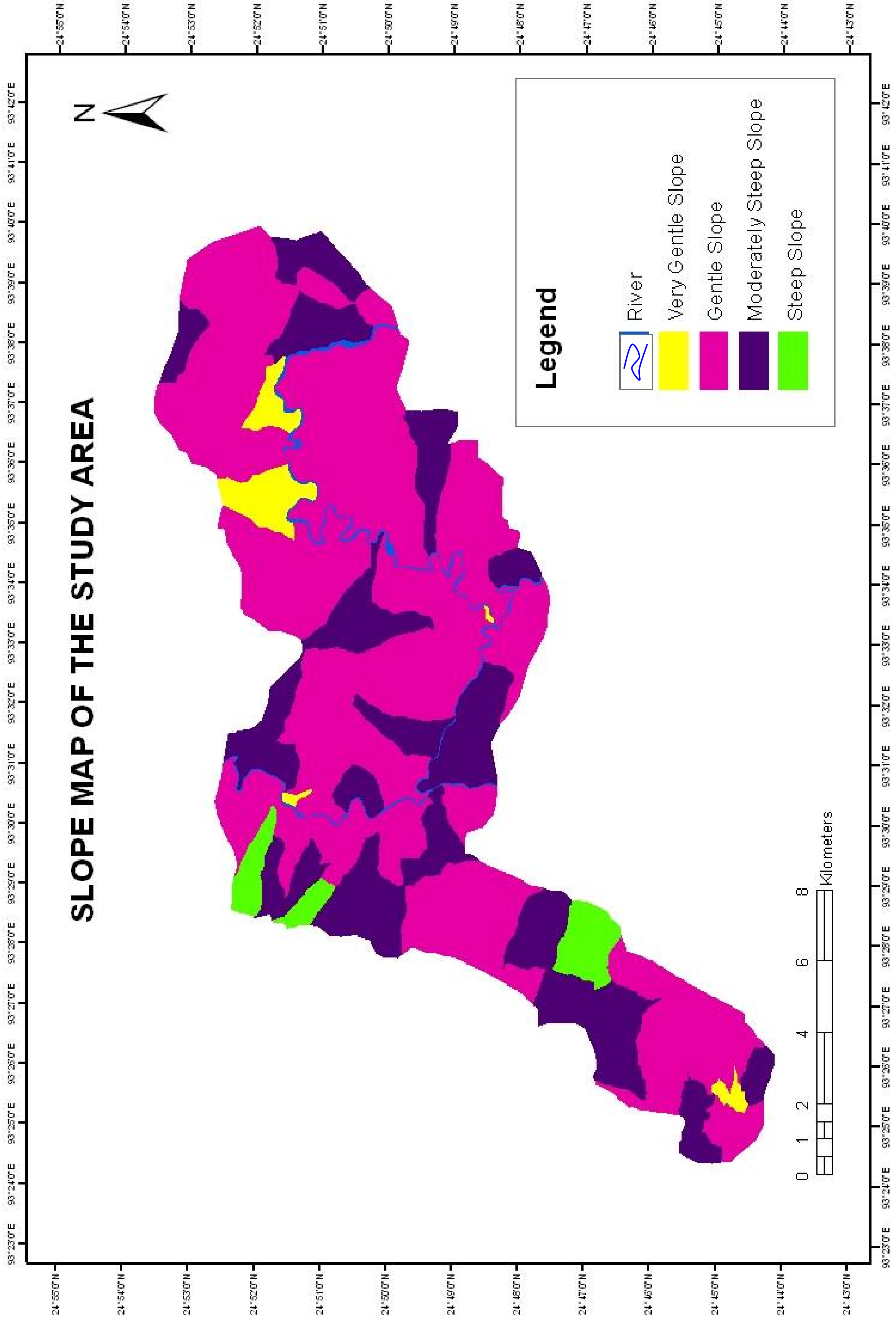
4.6 SLOPE MORPHOMETRY (SM)

. The slope morphometry map shall be prepared by dividing the larger topographical map into smaller units/facets within which the contour lines have the same standard spacing, i.e. the same number contour lines per kilometer of horizontal distance. Five categories of slope morphometry such as Escarpment / cliff slope, Steep slope, Moderately steep slope, Gentle slope and Very gentle slope are used depending on their slope angle in a particular facet. Various slope categories, based on the frequency of occurrence of particular angles of Slopes are shown in the map below. Since the slope angle is considered to be an important geo-environmental parameter inducing slope instability. LHEF value 2 (maximum) has assigned for it. The rating awarded for these sub-categories of slopes are furnished in the Table 5.

Table 5: Slope Morphometry (after Anbalagan, 1992 & BIS, 1998)

No. of contours per cm of length in the facet	Slope Angle	Category	LHEF Rating (Out of 2)	Area (Sq.Km.)	Percentage
< 7	< 15^0	Very gentle slope	0.5	**5.29**	**3.08**
8 – 12	16^0 - 25^0	Gentle slope	0.8	**108.70**	**63.16**
13 -18	26^0-35^0	Moderately steep slope	1.2	**5.90**	**3.43**
19 -25	36^0 -45^0	Steep slope	1.7	**48.22**	**28.02**
> 25	> 45^0	Escarpment / cliff slope	2.0	**--**	**--**

By studying the above table, it shows that the majority of the area falls under the gently slope category followed by steep slope, moderately steep slope and very gently slope covering an area of 108.70 sq.km, 48.22 sq.km,5.90 sq.km, and 5.29 sq.km respectively. Gently slope category covers almost half of the total area i.e., 63.16%, whereas very gentle slope covered the least i.e., only 5.29% of the whole area.


SLOPE MAP OF THE STUDY AREA
N
Legend
River
Very Gentle Slope
Gentle Slope
Moderately Steep Slope
Steep Slope
0 1 2 4 6 8
Kilometers

4.7 RELATIVE RELIEF

Relative Relief map represents the difference between the maximum and minimum heights within an individual facet. It shows the major breaks in the slopes of the area. Relief map of the study area and along parts of NH-53 has been prepared from the slope facet map by subtracting the lowest contour value from the highest contour value in a facet. The entire area has been divided into three categories of Relative Relief viz:

i) Low (<100m)

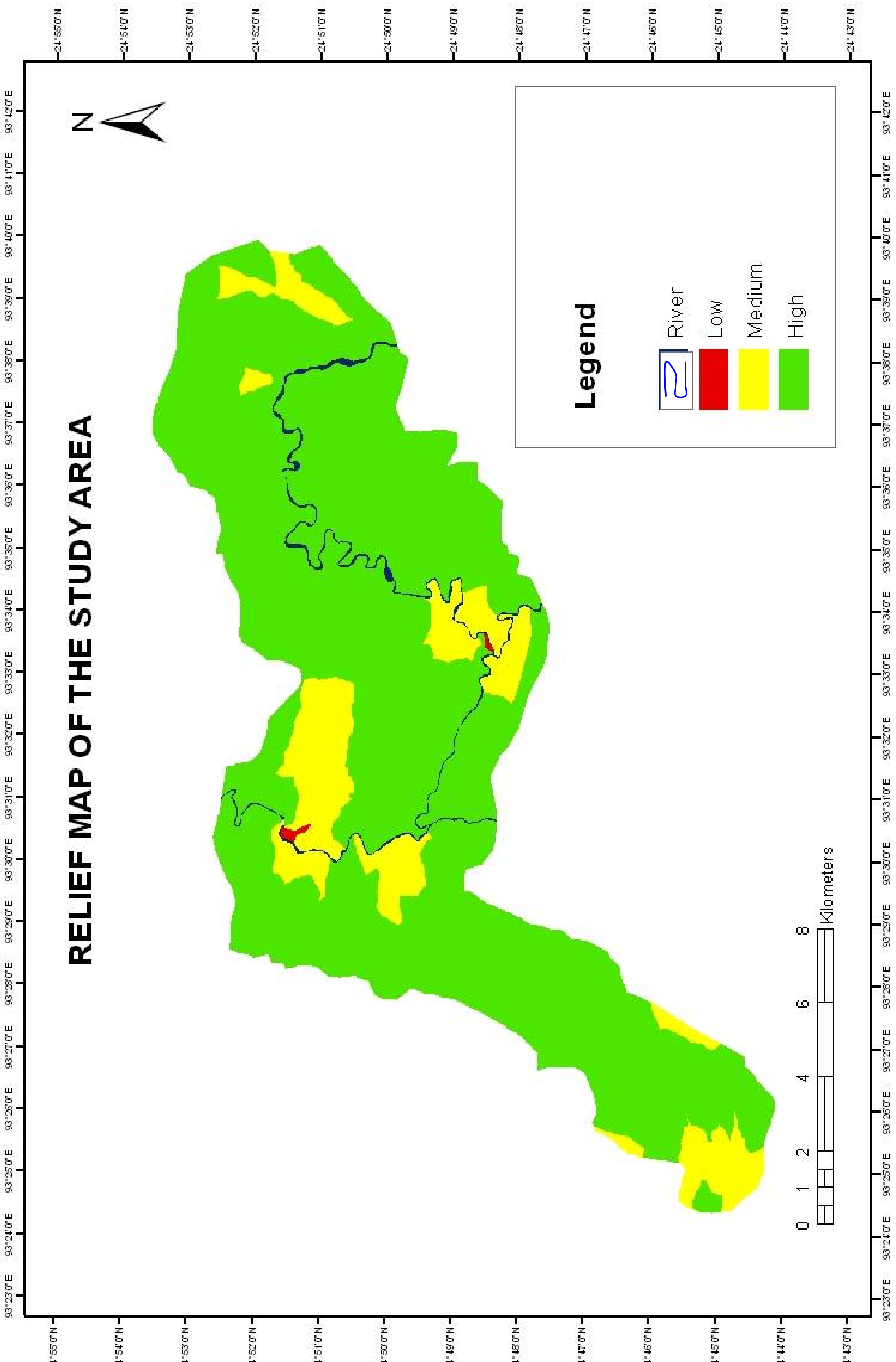
ii) Medium (101- 300m) and

iii) High (>300m).

The map shows that major part of the area lies under high relative relief followed by medium and low relative relief. The relative relief is considered as a geomorphic factor inducing landslide, where high relative indicates high slope height and more weight of slope forming materials in a facet. LHEF value 1 has been assigned to this geo-environmental parameter. LHEF rating for this factor are given in the Table 6.

Table 6: Relative Relief (after Anbalagan, 1992 & BIS, 1998)

Relative Relief	Category	LHEF Rating (Out of 1)	Area (Sq. Km.)	Percentage
< 100 m	Low relief	0.3	**0.18**	**0.11**
101 – 300 m	Medium relief	0.6	**23.95**	**13.92**
> 300 m	High relief	1.0	**145.61**	**84.61**


RELIEF MAP OF THE STUDY AREA
N
Legend
River
Low
Medium
High
0 1 2 4 6 8
Kilometers

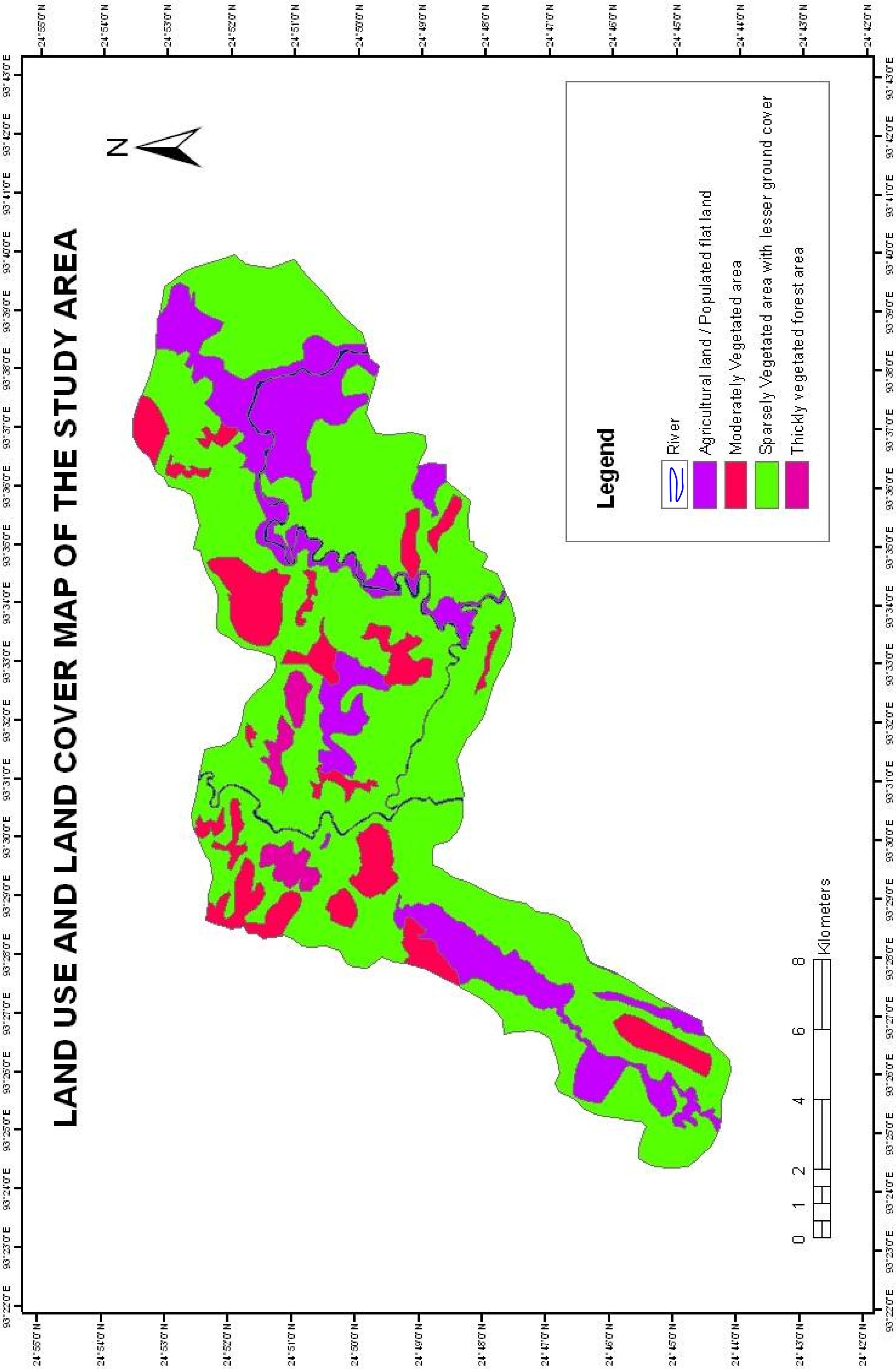
4.8 LAND-USE/LAND COVER

The nature of landuse/landcover is an indirect indication of the stability of hill slope. Vegetation cover generally checks the action of climatic agents and protects the slopes from erosion and weathering. Land-use/Land-cover map of the project area has been prepared on the basis of visual interpretation of LISS III with field check. On the basis of land-use/land-cover pattern, the area has been divided into five categories such as Agriculture land/ Populated flat land, Thickly Forest Cover, Moderately Forest Cover, Sparsely Forest Cover and Jhum/ Terrace cultivation/ Barren. Based on their intensity of vegetation cover, their relative ratings were awarded as shown in the table 7.

Table 7: Land use/ Land cover Rating Scheme (after Anbalagan, 1992 & BIS, 1998)

Category	LHEF Rating (Out of 2)	Area (Sq. Km.)	Percentage
Agriculture land/ Populated flat land	0.6	**31.28**	**18.18**
Thickly Forest Cover	0.85	**3.25**	**1.89**
Moderately Forest Cover	1.20	**20.85**	**12.12**
Sparsely Forest Cover	1.50	**114.36**	**66.45**
Jhum/ Terrace cultivation/ Barren	1.80	**--**	**--**

Land-use/land-cover map and table shows that the major part of the area falls under the sparsely forest cover having 66.45% of the total study area showing faster erosion and greater instability followed by agricultural land. Agriculture in general is practiced in low to very low slopes though moderately steep slopes are also used at some places. Since agriculture in the upside of the road represents the repeated artificial water charging for cultivation purpose may cause the instability of slope. Moderately forest cover and thickly forest cover were contributed in small percentage of about 12.12 and 1.89 respectively.


LAND USE AND LAND COVER MAP OF THE STUDY AREA
N
Legend
River
Agricultural land / Populated flat land
Moderately Vegetated area
Sparsely Vegetated area with lesser ground cover
Thickly vegetated forest area
0 1 2 4 6 8
Kilometers

4.9 HYDROLOGICAL CONDITION

Hydrogeological condition of a region is influenced by a number of factors like climate, lithology, structural discontinuities, neotectonic activities, landuse and landcover, drainage pattern/network etc. Groundwater condition of the study area (hilly terrain) is generally chanalised along structural discontinuity of rocks. It does not have uniform flow pattern. The observational evaluation of the groundwater on hill slopes is not possible over large scale. Thus, five categories of hydrological condition in LHEF rating scheme (Anbalagan, 1992 & BIS, 1998) such as Flowing, Dripping, Wet, Damp and Dry are used assigning with their respective rating. Southwestern portion of the study area is generally wet whereas northern portion is damp.

5. LANDSLIDE HAZARD EVALUATION FACTOR (LHEF) RATING SCHEME

LHEF rating scheme is a numerical weightage, governed by the major causative factors like lithology, structure, slope morphometry, relative relief, and land use/land cover of the slope instability. Each identified facet wise details of all these contributory factors was prepared for assigning Landslide Hazard Evaluation Factor (LHEF) rating for each factor. The maximum rating of an individual contributory factors is shown in table 8.

Table 8: Maximum Rating of an individual contributory factors

Contributory Factors	Rating (Maximum)
Lithology	2.0
Structure	2.0
Slope Morphometry	2.0
Relative Relief	1.0
Land use & Land Cover	2.0
Hydrological condition	1.0
Total	**10**

6. CALCULATION OF TOTAL ESTIMATED HAZARD (TEHD)

TEHD simply indicates the probabilities of instability of each facet.The Total Estimated Hazard (TEHD) of an individual facet has been calculated by adding the ratings of the individual causative factors obtained from the landslide hazard evaluation factor rating scheme. Depending on the value obtained from the total estimation hazard, each facet falls into their respective hazard classification.

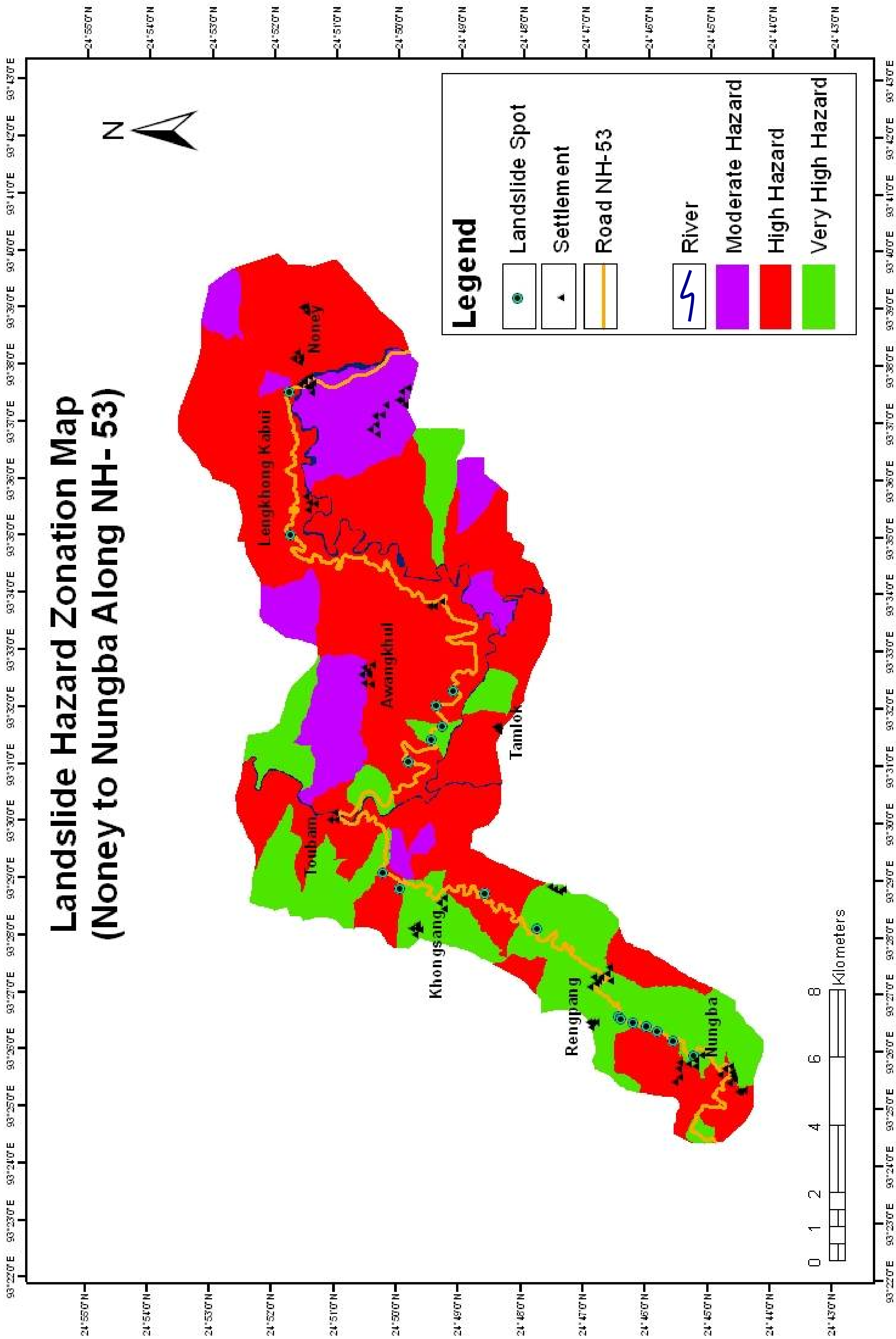
7. LANDSLIDE HAZARD ZONATION

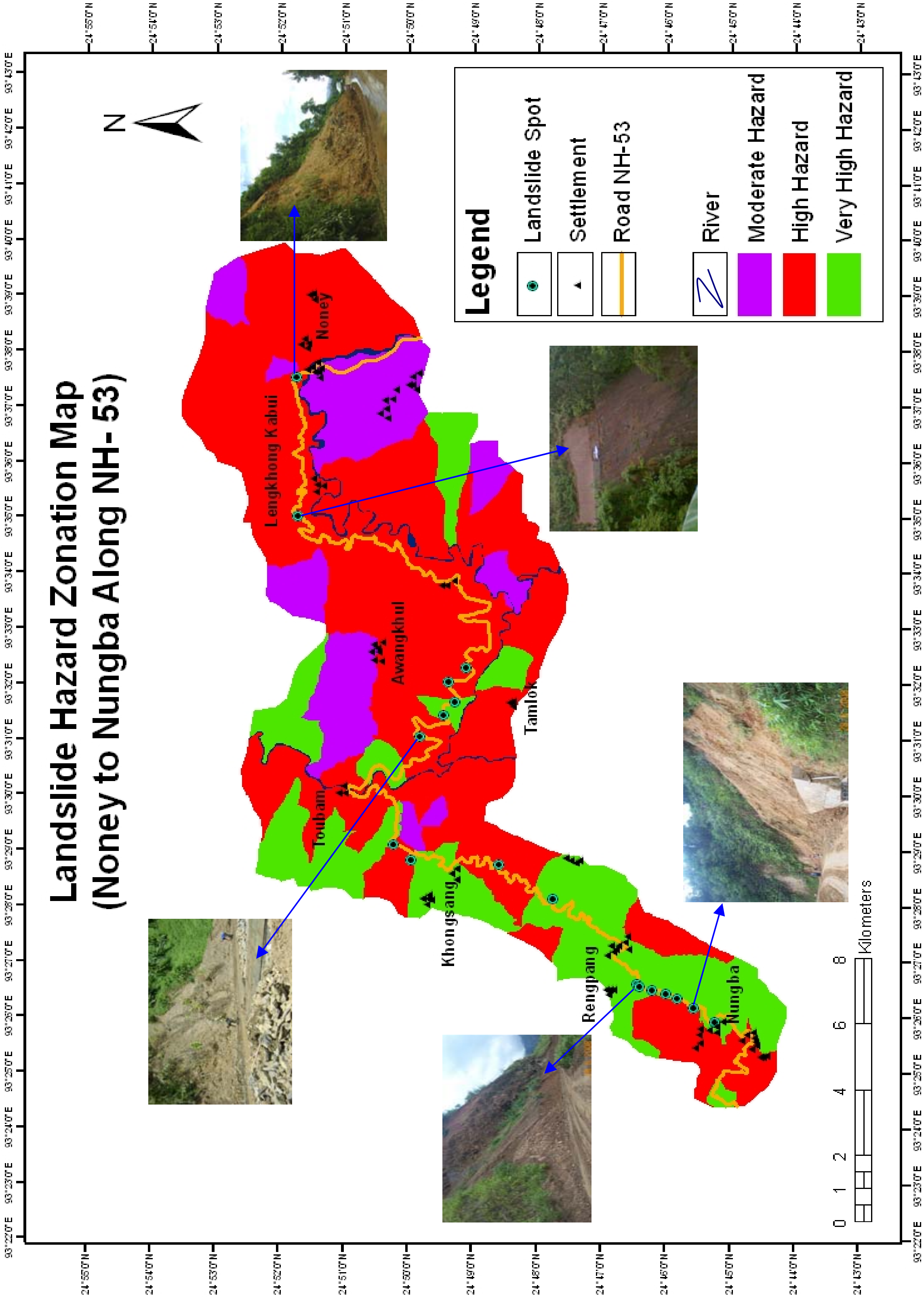
Based on the distribution of TEHD values of each facet the landslide hazard zonation map has been prepared and facilitates spatial classification of the study area into three zones viz. Moderate Hazard (MH), High Hazard (HH) and Very High Hazard (VHH).

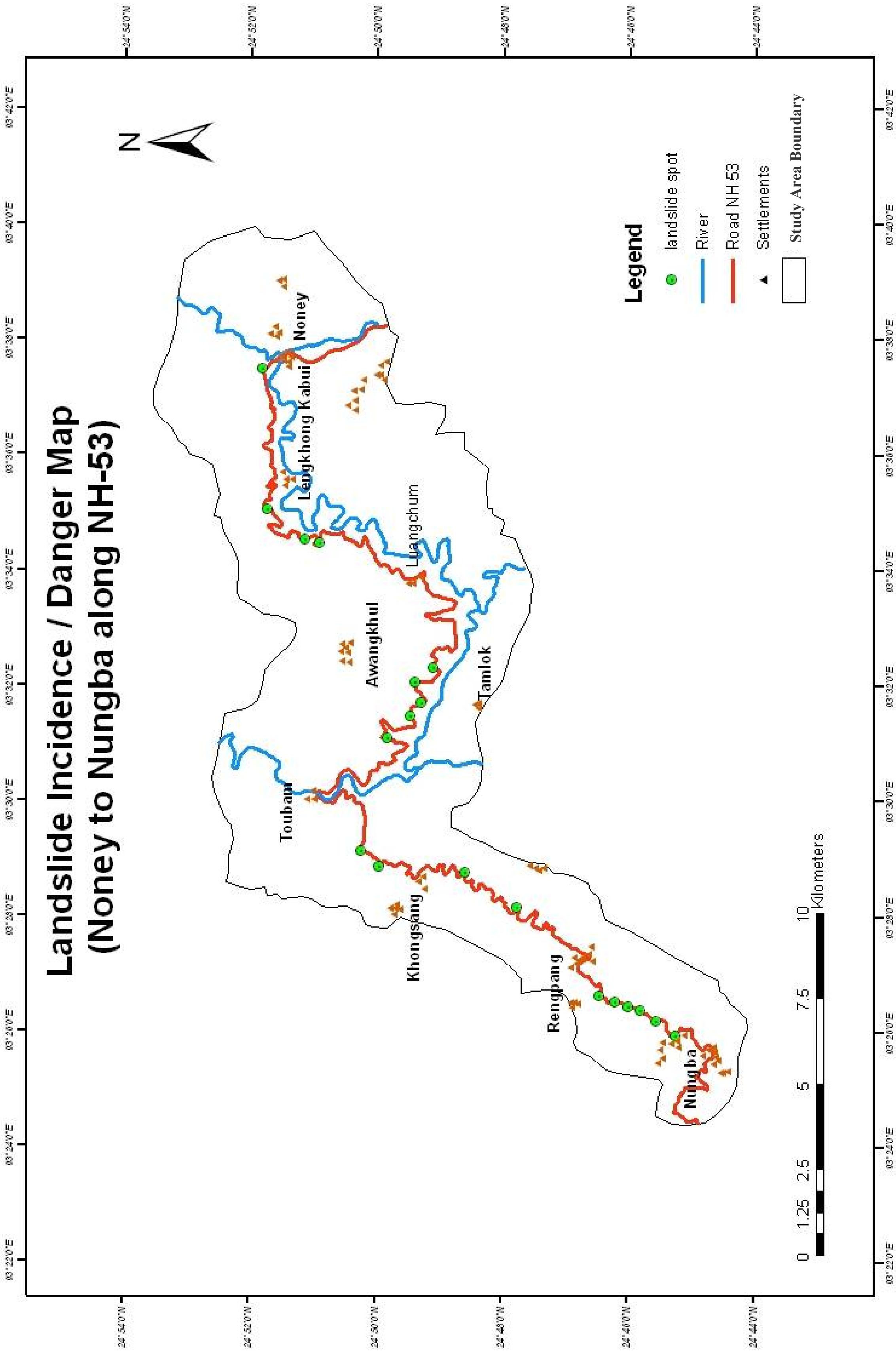
Table 9: LHZ based on TEHD (after Anbalagan, 1992 & BIS, 1998)

Zone	TEHD Value	Description of Zones	Area (Sq. Km.)	Percentage
I	5.1 – 6.0	**Moderate Hazard (MH) Zone**	**25.2**	**14.57**
II	6.1 – 7.5	**High Hazard (HH) Zone**	**104.8**	**60.58**
III	> 7.5	**Very High Hazard (VHH) Zone**	**40.2**	**23.24**

From the above table the study area is devoid of Very Low Hazard Zone (VLH) and Low Hazard Zone (LH). Major part of the area falls into the category of High Hazard Zone which is about 60.58% of the total area covering 104.8 sq.km. followed by Very High Hazard Zone covering 40.2 sq.km with 23.24% of the total area. 25.2 sq. km. covering Moderate Hazard with 14.57% of the total area respectively. Majority of the study area are distributed with High Hazard and Very High Hazard zone unevenly. Landslide hazard zonation map has been prepared by assigning the relative numerical weightages of each facet incorporating from various thematic maps like lithology, structure, slope, relief, land use/land cover and hydrology etc. through Geographic Information System (GIS) to validate their respective hazard zones. Landslide Hazard Zonation map and Landslide Incidence map of the study area are shown in the maps below.


Landslide Hazard Zonation Map
(Noney to Nungba Along NH- 53)
N
Legend
Landslide Spot
Settlement
Road NH-53
River
Moderate Hazard
High Hazard
Very High Hazard
Noney
Lengkhong Kabui
Awangkhul
Tamlon
Toubam
Khongsang
Rengpang
Nungba
0 1 2 4 6 8 Kilometers


Landslide Hazard Zonation Map
(Noney to Nungba Along NH- 53)
N
Legend
Landslide Spot
Settlement
Road NH-53
River
Moderate Hazard
High Hazard
Very High Hazard
Noney
Lengkhong Kabui
Awangkhul
Tamlon
Toubam
Khongsang
Rengpang
Nungba
0 1 2 4 6 8 Kilometers


Landslide Incidence / Danger Map
(Noney to Nungba along NH-53)
N
Legend
landslide spot
River
Road NH 53
Settlements
Study Area Boundary
Noney
Leingkhong Kabui
Luangchum
Awangkhul
Tamlok
Toubam
Khongsang
Rengpang
Nungba
0 1.25 2.5 5 7.5 10
Kilometers

8. LANDSLIDE OCCURRENCES

Table 11: Profile of the Landslide occurrences

Landslide Spot	Name of Village	Location	Type of slide	Lithology	Probable Causes
1	**Nungba**	**93°25′54"E 24°45′14"N**	**D.F.**	Intercalations of shale and bedded sandstone	**N**
2	**Raungdai 1**	**93°26′10"E 24°45′34"N**	**R.F + D.F.**	Intercalations of shale and bedded sandstone	**N + A**
3	**Raungdai 2**	**93°26′21"E 24°45′49"N**	**R.F + D.F.**	Intercalations of shale and bedded sandstone	**N + A**
4	**Raungdai 3**	**93°26′25"E 24°46′0"N**	**R.F + D.F.**	Intercalations of shale and bedded sandstone	**N + A**
5	**Raungdai 4**	**93°26′29"E 24°46′12"N**	**R.F + D.F.**	Intercalations of shale and bedded sandstone	**N + A**
6	**Raungdai 5**	**93°26′33"E 24°46′24"N**	**R.F + D.F.**	Intercalations of shale and bedded sandstone	**N + A**
7	**Raungdai 6**	**93°26′36"E 24°46′27"N**	**R.F + D.F.**	Intercalations of shale and bedded sandstone	**N + A**
8	**Rengpang**	**93°28′07"E 24°47′46"N**	**D.F.**	thickly bedded to massive sandstones with sandy-shales and silty-shales intercalations.	**N**
9	**Khongsang 1**	**93°28′44"E 24°48′35"N**	**D.F.**	Intercalations of shales, siltstones and sandstones cover with recent alluvium soil	**N + A**
10	**Khongsang 2**	**93°28′50"E 24°49′58"N**	**D.F.**	Intercalations of shales, siltstones and sandstones cover with recent alluvium soil	**N + A**
11	**Khongsang 3**	**93°29′06"E 24°50′15"N**	**D.F.**	Intercalations of shales, siltstones and sandstones cover with recent alluvium soil	**N + A**
12	**Awangkhul Part II**	**93°31′03"E 24°49′51"N**	**D.F. + M.S.**	silty and sand mixed with some clay	**N + A**
13	**Awangkhul 1**	**93°31′26"E 24°49′28"N**	**D.F.**	silty and sand mixed with some clay	**N**
14	**Awangkhul 2**	**93°31′40"E 24°49′18"N**	**D.F.**	silty and sand mixed with some clay	**N**
15	**Awangkhul 3**	**93°32′02"E 24°49′24"N**	**D.F.**	silty and sand mixed with some clay	**N**
16	**Awangkhul 4**	**93°32′17"E 24°49′07"N**	**D.F.**	silty and sand mixed with some clay	**N**
17	**Rongkhong bridge**	**93°35′01"E 24°51′45"N**	**D.F.**	Debris made up of sandy & clays	**A**
18	**Khumji**	**93°37′32"E 24°51′46"N**	**D.F.**	Debris made up of sandy & clays	**A**
D.F. – Debris Fall, R.F. – Rock fall, M.S. – Mud Slide, N – Natural, A - Anthropogenic					

9. CAUSATIVE FACTORS OF EACH FACET

Table 11: Causative factors of each facet

Facet	CAUSATIVE FACTOR OF EACH FACET						TEHD	HAZARD CATEGORY
	Lithology	Structure	Slope	Relief	Land_use	Hydrology		
1	2	1.75	0.5	1	0.8	0.2	6.25	HH
2	2	1.65	0.8	1	0.6	0.1	6.15	HH
3	2	1.65	0.8	0.6	0.6	0.2	5.85	MH
4	2.2	1.75	0.8	1	1.05	0.2	7	HH
5	2	1.75	0.8	1	1.1	0.2	6.85	HH
6	2	1.75	0.5	1	1.5	0.1	6.85	HH
7	2	1.75	0.8	1	1.35	0.2	7.1	HH
8	2	1.75	0.8	1	1.1	0.2	6.85	HH
9	2	1.35	0.8	1	0.6	0.2	5.95	MH
10	2.2	1.75	0.8	0.6	1.5	0.2	7.05	HH
11	2	1.65	0.8	1	1.5	0.2	7.15	HH
12	2	1.45	0.5	0.3	1.5	0.2	5.95	MH
13	2	1.75	0.8	1	1.1	0.2	6.85	HH
14	2	1.35	0.8	1	1.35	0.35	6.85	HH
15	2	1.35	0.8	1	1.05	0.5	6.7	HH
16	2	1.35	1.2	1	1.5	0.5	7.55	VH
17	2	1.35	1.2	1	1.05	0.5	7.1	HH
18	2.2	1.35	0.8	1	1.5	0.5	7.35	HH
19	2	1.35	0.8	1	1.1	0.5	6.75	HH
20	2	1.35	1.2	1	1.5	0.5	7.55	VH
21	2.2	1.35	1.2	1	1.5	0.5	7.75	VH
22	2	1.35	0.8	1	1.05	0.5	6.7	HH
23	2	1.35	0.8	0.6	1.5	0.5	6.75	HH
24	2	1.35	0.5	0.3	1.2	0.5	5.85	MH
25	2	1.35	0.8	0.6	0.6	0.5	5.85	MH
26	2	1.35	1.2	1	1.5	0.5	7.55	VH
27	2	1.35	0.8	0.6	0.6	0.5	5.85	MH
28	2	1.35	1.2	1	1.5	0.5	7.55	VH
29	2	1.75	1.2	1	0.6	0.1	6.65	HH
30	2	1.75	0.8	1	1.5	0.1	7.15	HH
31	2	1.75	1.2	1	1.5	0	7.45	HH
32	2	1.75	0.8	0.6	1.5	0	6.65	HH
33	2	1.75	1.2	1	1.5	0	7.45	HH
34	2	1.75	1.2	0.6	1.5	0	7.05	HH
35	2	1.45	0.8	1	0.6	0.1	5.95	MH
36	2	1.75	0.8	1	1.5	0.1	7.15	HH
37	2.2	1.75	1.2	1	1.05	0.2	7.4	HH
38	2	1.4	0.8	1	0.6	0.2	6	MH
39	2	1.75	1.2	1	1.5	0	7.45	HH
40	2	1.75	0.8	0.6	0.6	0	5.75	MH
41	2	1.75	0.8	1	1.5	0.1	7.15	HH
42	2	1.75	0.8	1	1.2	0.2	6.95	HH
43	2	1.4	0.8	1	0.6	0.2	6	MH
44	2.4	1.35	1.2	1	1.5	0	7.45	HH
45	2.4	1.35	1.2	1	1.5	0.1	7.55	VH
46	2.4	1.35	0.8	1	1.35	0.5	7.4	HH
47	2.4	1.35	0.8	0.6	1.5	0.5	7.15	HH
48	2.4	1.35	0.8	1	1.5	0.2	7.25	HH
49	2.4	1.35	1.2	1	1.35	0.1	7.4	HH
50	2.4	1.35	0.8	0.6	1.2	0.2	6.55	HH
51	2.4	1.35	0.8	1	1.35	0.5	7.4	HH
52	2.4	1.35	0.8	1	1.15	0.5	7.2	HH
53	2.4	1.35	0.8	0.6	1.15	0.5	6.8	HH
54	2.4	1.35	1.2	1	1.2	0.5	7.65	VH

55	2.4	1.35	0.8	1	1.17	0.5	7.22	**HH**
56	2.4	1.35	1.2	1	1.5	0.35	7.8	**VH**
57	2.4	1.35	1.7	1	1.2	0.5	8.15	**VH**
58	2.4	1.35	1.2	1	1.35	0.2	7.5	**HH**
59	2.4	1.35	1.2	1	1.35	0.2	7.5	**HH**
60	2.4	1.35	1.2	1	1.5	0.1	7.55	**VH**
61	2.4	1.35	0.8	0.6	0.6	0.2	5.95	**MH**
62	2.4	2.05	0.8	1	1.5	0.1	7.85	**VH**
63	2.4	2.05	0.8	1	1.05	0.1	7.4	**HH**
64	2	1.35	0.8	0.6	0.8	0.2	5.75	**MH**
65	2.4	1.35	1.2	1	1.17	0.5	7.62	**VH**
66	2.4	2.05	1.2	1	1.05	0.1	7.8	**VH**
67	2.4	2.05	0.8	1	1.1	0.2	7.55	**VH**
68	2.4	2.05	0.8	1	1.05	0.1	7.4	**HH**
69	2.4	2.05	1.2	1	1.05	0.1	7.8	**VH**
70	2.4	2.05	0.8	1	1.05	0.1	7.4	**HH**
71	2.4	2.05	0.8	0.6	1.5	0	7.35	**HH**
72	2.4	2.05	1.2	1	1.5	0.1	8.25	**VH**
73	2.4	2.05	0.8	0.6	1.5	0.2	7.55	**VH**
74	2.4	2.05	1.2	1	0.6	0.2	7.45	**HH**
75	2.4	2.05	0.8	1	1.1	0.2	7.55	**VH**
76	2.4	2.05	0.8	1	1.05	0.1	7.4	**HH**
77	2.4	2.05	1.2	0.6	1.05	0.1	7.4	**HH**
78	2.4	2.05	0.8	0.6	1.5	0	7.35	**HH**
79	2.4	2.05	0.8	0.6	1.05	0.1	7	**HH**
80	2.4	2.05	1.2	1	1.5	0.2	8.35	**VH**
81	2.4	1.75	0.5	0.6	0.6	0.2	6.05	**HH**
82	2.4	2.05	0.8	0.6	1.05	0.1	7	**HH**
83	2.4	2.05	1.2	1	1.5	0	8.15	**VH**
84	2.4	2.05	1.2	1	1.5	0.1	8.25	**VH**
85	2.4	1.35	1.2	1	1.5	0.5	7.95	**VH**
86	2	1.35	1.2	1	1.5	0.5	7.55	**VH**
87	2	1.75	0.8	1	1.35	0.1	7	**HH**
88	2.2	1.75	0.8	1	1.05	0.2	7	**HH**
89	2	1.75	0.8	1	1.2	0.2	6.95	**HH**
90	2.4	1.35	0.8	0.6	1.5	0.2	6.85	**HH**
91	2	1.75	0.8	1	1.5	0.1	7.15	**HH**
92	2	1.75	0.8	1	1.05	0.1	6.7	**HH**
93	2	1.75	1.2	1	1.5	0.2	7.65	**VH**
94	2.4	1.35	1.7	1	1.35	0.5	8.3	**VH**
95	2.4	2.05	1.7	1	1.05	0.1	8.3	**VH**
96	2.4	1.35	1.2	1	1.5	0	7.45	**HH**
97	2	1.75	0.8	1	1.5	0	7.05	**HH**
98	2	1.75	0.8	1	1.5	0	7.05	**HH**
99	2	1.75	0.8	0.6	1.5	0	6.65	**HH**
100	2	1.75	0.8	1	1.5	0	7.05	**HH**
101	2	1.75	1.2	1	1.1	0.2	7.25	**HH**
102	2	1.75	1.2	1	1.35	0.2	7.5	**HH**
103	2	1.45	0.8	1	0.6	0.1	5.95	**MH**
104	2	1.4	0.8	1	0.6	0.2	6	**MH**
105	2.4	1.35	0.8	1	1.35	0.5	7.4	**HH**
106	2.4	2.05	0.8	1	1.35	0.2	7.8	**VH**
107	2	1.35	0.8	0.6	0.6	0.5	5.85	**MH**
108	2.4	1.35	0.8	1	1.5	0.1	7.15	**HH**
109	2.4	2.05	0.8	1	1.1	0.2	7.55	**VH**

TEHD – Total Estimated Hazard, MH – Moderate Hazard, HH – High Hazard, VH – Very High Hazard.

10. GEOTECHNICAL INVESTIGATION OF THE TWO SLIDES

Geotechnical investigations of the slope instability of the slide have to be carried out in a systematic way in order to account for all the parameters responsible for instability. The investigation has been carried out through both laboratory and field studies. Some of the common approaches are Direct Shear Test, Point Load Test, Uni-axial Compressive Test (UCS), Tri-axial Shear Test, Bieniawski's (1989) Rock Mass Rating (RMR), Romana's (1985) Slope Mass Rating (SMR), and Three Phase Analysis of Soil etc. Identification of mode of failure is important to choose relevant analytical method for investigation. Depending upon the mode of failure, nature of the slide, structures and discontinuity, rock and soil types, the relevant approaches has been chosen. Groundwater plays an important role for assessment of the stability of the rock slope and reduction of the stability.

11. DETERMINATION OF FACTOR OF SAFETY (F) USING CIRCULAR FAILURE CHARTS OF AWANGKHUL PART II SLIDE (DIRECT SHEAR TEST APPROACH)

Generally landslides occur when the disturbing/driving force (FD), which is chiefly resulted from the self weight of the slope forming materials exceeds the resisting force (FR) given by the shear strength of the materials. So, the factor of safety of a slope is the ratio of resisting forces to driving forces, **i.e. F = resisting forces / driving forces**

If the factor of safety is less than or equal to 1 (i.e., $F \leq 1$), the slope will fail because driving forces will equal or exceed the resisting forces. If F is significantly greater than 1, the slope will be quite stable. However, if F is slightly greater than 1, small disturbances may cause the slope to fail. For example, if $F = 2$, the slope has resisting forces twice as large as the driving forces, and it will be extremely stable. If, on the other hand, $F = 1.05$, the slope's strength is only 5% greater than the driving forces, and slight undercutting or steepening, or very heavy rain, or seismic shaking may easily cause it to fail.

Interestingly, during rainy season, the driving force, FD is maximum and resisting force, FR is minimum. And so, landslides are common during rainy seasons. Similarly in Manipur also landslides are common during rainy season especially in the months of July-August along the hilly tracts of NH-53.

The present approach i.e. Direct Shear Test deals with the determination of factor of safety of Awangkhul Part II landslide for the investigation of the slope instability of the sliding area. Direct shear test is generally done in-situ soil sample in laboratory. If the slope is dominantly consisted of fine fraction (maximum upto sand size) which is homogenous in nature, shear strength parameters estimated from this test gives nearly accurate value. Even if disturbed samples are tested, the values obtained may not differ much from the former case. If the slope is consisted of debris, it contains a mixture of finer materials upto sand size fraction and coarser fractions more than gravel size. In this case, the test is carried out on samples of finer fraction omitting the coarser fractions. In this case, the values obtained may not be true representative of field condition. Hence a careful visual estimate is required to get the percentage of coarser fraction. This is because coarser fraction, if well disturbed among finer particles, offers resistance to failure path through the slope materials and hence increases its shear strength. So, a judicial judgement is required to increase the value of shear strength parameters by 10-30% depending upon the amount of coarser fraction present. The following input parameters are used for the calculation of factor of safety.

11.1 Average Slope angle

It is the average angle between horizontal surface and slope face where sliding occurs. It can be obtained from field observation.

11.2 Height of the slope (H)

It is the vertical height of the slope face measured from the toe of the slope upto highest point of phreatic surface. Generally it is represented by H.

11.3 Unit weight of the soil (γ)

It is defined as the weight per unit volume. Hence it will be represented in terms of kN/m^3

Thus,

$$\gamma = \frac{Weight\ of\ \ the\ soil}{Volume\ of\ \ the\ soil}(\kappa N / m)$$

$$= \ Bulk\ Density(\rho)\ \times\ 9.81(\kappa N / m)$$

Table 12: Unit weight (γ) of the Awangkhul Part II soil sample

Sl.No.	Location	Sample No.	Type of Soil	Bulk Density (ρ) gm/cc	Unit Weight (γ) N/ m^3
1	Awangkhul Part II	1	Undisturbed	1.743	17099
2	-do-	2	Undisturbed	1.718	16854
3	-do-	1	Disturbed	1.805	17707
4	-do-	2	Disturbed	1.856	18207

11.4 Moisture Content (W)

The difference in weight between wet soil and dry soil gives the moisture content of the soil sample. Mathematically it can be expressed as

$$W = \frac{W2 - W3}{W3 - W1} \times 100\%$$

Where, W = Moisture content
$W1$= Weight of the container (including lid)
$W2$= Weight of the container (including lid) and wet soil
$W3$= Weight of the container (including lid) and dry soil

Table 13: Moisture content (%) of the Awangkhul Part II soil sample

Sl.No.	Location	Sample No.	Type of Soil	Moisture Content (%)
1	Awangkhul Part II	1	Undisturbed	20.06
2	-do-	2	Undisturbed	20.81
3	-do-	3	Undisturbed	21.13
4	-do-	1	Disturbed	27.09
5	-do-	2	Disturbed	25.56
6	-do-	3	Disturbed	26.12

11.5 Cohesion (c)

Cohesion is the innate "stickiness" of a material, the attraction of its molecules for each other. For example, clay and granite are both cohesive. Dry sand, on the other hand is cohesionless, that is, its cohesion is zero.

11.6 Angle of internal friction (ϕ)

Internal friction is due to the grains of the material rubbing against each other. The friction depends on:-

1) how slick the grains are (the coefficient of friction or angle of internal friction), which depends on the particular material, and 2) how hard the grains are being forced against each other by gravity (the normal stress). If there is water in the pore spaces between the grains, the water pressure forces the grains apart and reduces the frictional strength. Note that this is not lubrication. Rather than making things slicker, the increased pore pressure reduces the normal stress (reduces how hard the grains are forced together), thus reducing the frictional strength.

As an equation:

Internal friction = coefficient of friction x (normal stress - pore pressure)

All the strength of dry sand comes from internal friction.

12. LOCATION OF LANDSLIDE SITE

A circular type of landslide has been occurred at Awangkhull Part-II (**93^{0}31′15″ E and 24^0 49′29″ N**) along **NH-53** which is about **26** km away from Noney with an area of about **0.021 sq.km**. This type of failure often occurs on hill slopes characterized by overburden soil and debris. The landslide damaged the road over a distance of about 100m, where it slides down by a vertical height of about 86m. The slide has been observed on 17th September 2008 during field survey.

Landslide at Awangkhul Part-II

12.1 Local Geology and Causes of Slide

The rocks in the slide area are occupied by the **Barail Group** of rocks. The Barail Group are characterized by alteration of sandstones and shale showing intercalations sometimes thickly bedded sandstone beds showing typical turbidite character at places. However the landslide occurred on a slope about 35^0 predominantly consisting of soil debris accumulated on the surface of the hills. The major reason of the landslide is both **anthropogenic** and **natural** causative factors.

Closure view of the Slide

12.2 Type of Slope Forming Materials

The slope forming materials predominantly consisting of debris made up of silty and sand mixed with some clay along with very small amount of rock debris. Geological plan map and section of the slide area in **1:2000** scale is shown in fig. 3 & 4.

Collection of Soil Samples

GEOLOGICAL MAP OF AWANGKHUL PART II

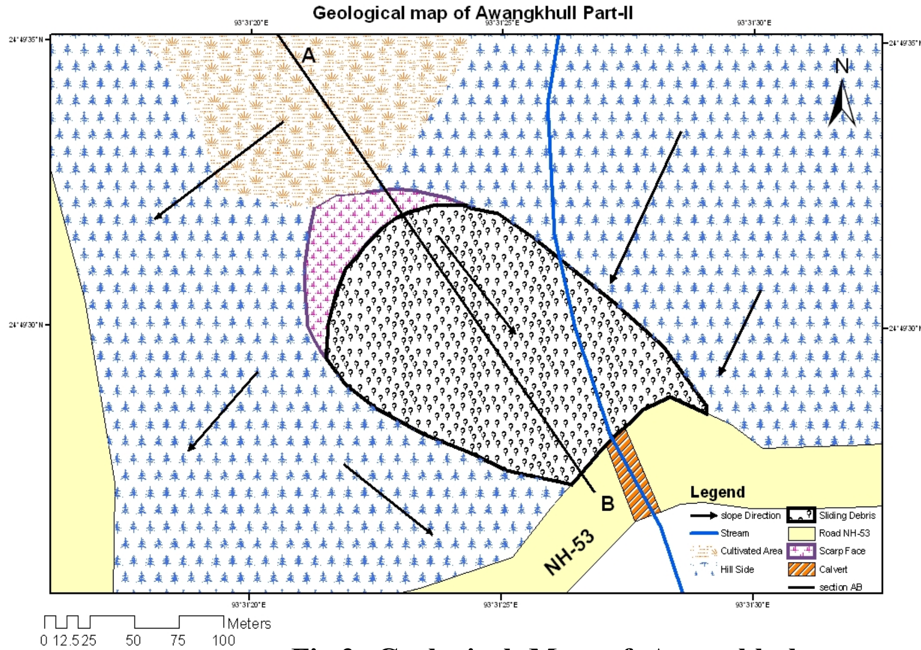


Fig 3: Geological Map of Awangkhul

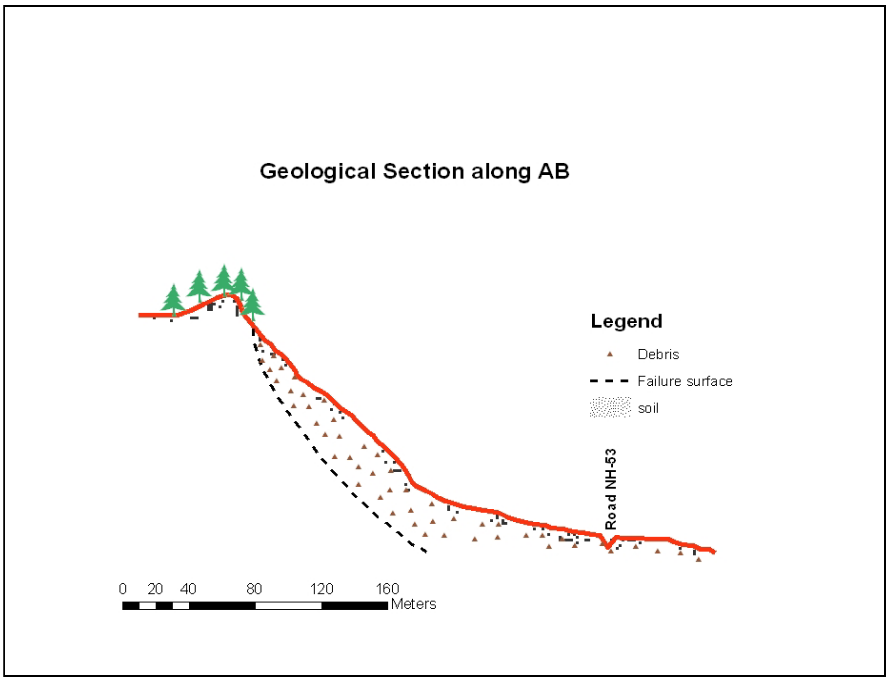


Fig 4: Geological Section along AB

13. STRESS STRAIN CURVE

Stress Strain curve of the soil samples of the landslide site are made by plotting the data of displacement against load calibrating from Direct Shear Test

Analysis of Soil Sample (Direct Shear)

14. FAILURE ENVELOPE CURVE

Normal stress (σ) and Shear stress (τ) parameters of the soil samples of the slide site can be obtained from stress strain curve of soil samples by taking the highest peak point from the load-displacement curve. Some of parameters are shown in the table 14.

Sl. No.	Normal Stress, σ (kN/m^2)	Shear Stress, τ (kN/m^2)
1	49	58.89
2	98	97.23
3	49	55
4	19.6	52
5	29.4	38.8
6	9.8	25.27
7	14.7	25.5

Table 14: Normal stress & Shear stress parameters of soil sample of Awangkhul Part-II.

Then Normal stress (σ) and Shear stress (τ) parameters are plotted on the Normal stress & Shear stress graph to obtain the value of cohesive strength (C) and internal frictional angle (ϕ) of the soil sample of slide site.

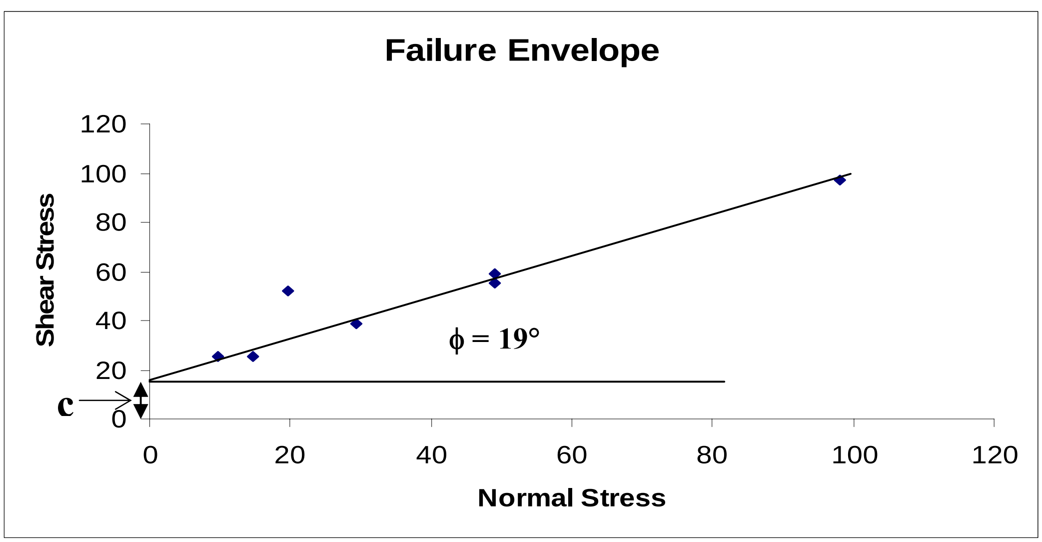


Fig 5 : Normal stress & Shear stress graph of soil samples of Awangkhul Part-II

15. DETAILED OF INPUT PARAMETERS CONSIDERED IN ANALYSIS

Average Slope angle - 35°

Height of the slope (H) – 86m

Unit weight of soil (γ) – 17957 N/m^3

Cohesion (c) – 16,000 N/ m^2

Angle of internal friction (ϕ) – 19°

Moisture content – 20.81 %

16. ANALYSIS OF AWANGKHUL PART-II SLIDE USING CFCs

Determination of Factor of Safety (F) of Awangkhul Part-II slide has been carried out using following steps.

Step 1

The soil samples were collected from three different heights of slided mass. They were mixed thoroughly to get a collective representative of the entire mass. In laboratory, this was again divided into three equal proportions and weight of individual part was noted. Then they were heated in hot oven for 24 hours after which their individual weight was again noted. The difference in weight gave moisture content of the samples and this value could be used to choose a particular chart to be considered for groundwater flow condition. The moisture content of this area ie. Awangkhul Part-II is **20.81**. So, the Circular Failure Chart of **25%** groundwater condition has been choosing for the derivation of Factor of safety (F).

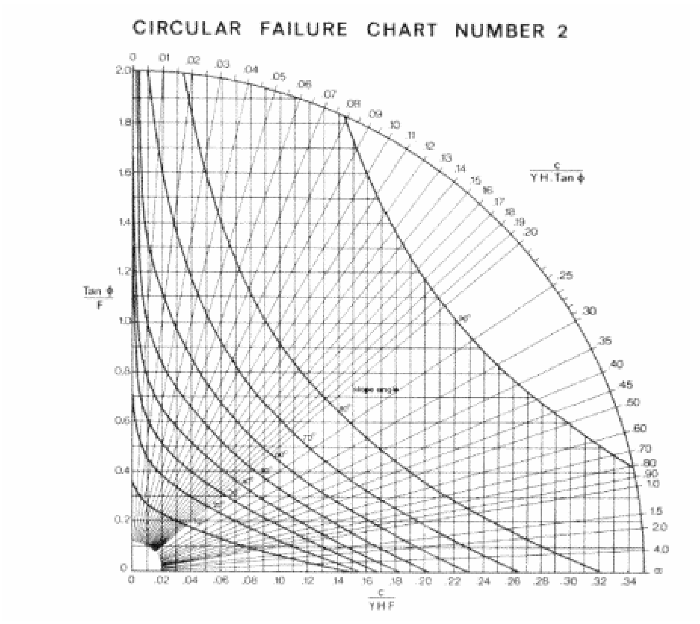


Fig 6: Circular Failure Chart for 25% groundwater condition

Step 2

The value of dimensionless ratio ($C/\gamma.H.\tan\phi$) = 0.03, from the data obtained from field observations and tests conducted, is calculated.

Step 3

This value is marked on the peripheral arc (outer circular scale) of the failure chart for the corresponding groundwater condition. The radial line from the outer circular scale is then followed to the particular curve for slope angle (40° as in this case).

Step 4

The corresponding value of $\tan\phi/F$ (Y-intercept) and $C/\gamma.H.$ (X-intercept) is found out by projecting horizontally and vertically on two axes of the chart. Hence the F value is calculated as average of the above two F values (obtained from X & Y intercepts).

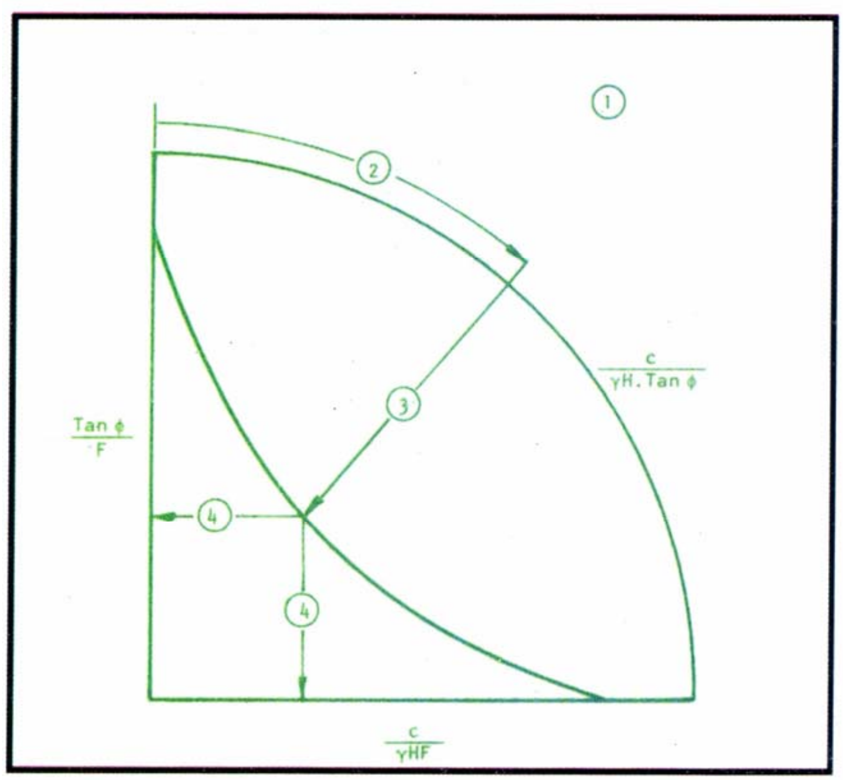


Fig 7: Calculation of Factor of safety from Circular Failure Chart (Hoek & Bray, 1981)

From the circular failure chart we have the Y- intercept value is 0.62 (approx.) and putting the value of tan ϕ we get,

$$\text{Tan } \phi / F = 0.625$$

$$\text{Or,} \quad F = \tan 19° / 0.6$$

$$= 0.3443/0.6$$

$$= 0.57$$

Similarly obtaining X- intercept value of 0.025 (approx.) and putting the values of C, γ and H we get,

$$C/\gamma .H.F = 0.019$$

$$F = C/ 0.025 \text{ x } \gamma \text{ x } H$$

$$= 16{,}000/0.019 \text{ x } 17957 \text{ x } 86$$

$$= 0.54$$

$$\mathit{Factor\ of\ Safety} = \frac{\text{F value along Y - intercept} + \text{F value along X - intercept}}{2}$$

$$= \frac{0.57 + 0.54}{2}$$

$$= 0.55$$

17. SLOPE INSTABILITY OF RAUNGDAI LANDSLIDE – (SMR APPROACH)

17.1 LOCATION OF LANDSLIDE SITE

A landslide has been occurred at Raungdai/Blongdai (**$93^0 26'29''$ E and $24^0 46'19''$ N**) along **NH-53** which is about **4** km to Nungba. The area is characterized by intercalations of shale and bedded sandstone. The general slope is N 129° strike and 36° S Dip. The landslide damaged the road over a distance of about 120m, where it slides down by a vertical height of about 100m. The slide has been observed on 31st December 2008 during field survey.

Landslide at Raungdai

17.2 TYPE OF FAILURE

After kinematics analysis of the orientations of discontinuities and field observations of the sliding site, the Raungdai/Blongdai landslide has been identified as Wedge failure. Wedge failure is a type of transitional failure that occurs on rock slopes. When geological discontinuities are unfavorably oriented with reference to general slope. In case of wedge failures, two discontinuities strike obliquely across the slope face with their line of intersection getting day lighted on the slope and the rock wedge formed due to this intersection will tend to slide down along the line of intersection, provided that the plunge of line of intersection is less than the slope inclination and greater than the angle of internal friction.

Landslide at Raungdai

GEOLOGICAL MAP OF RAUNGDAI

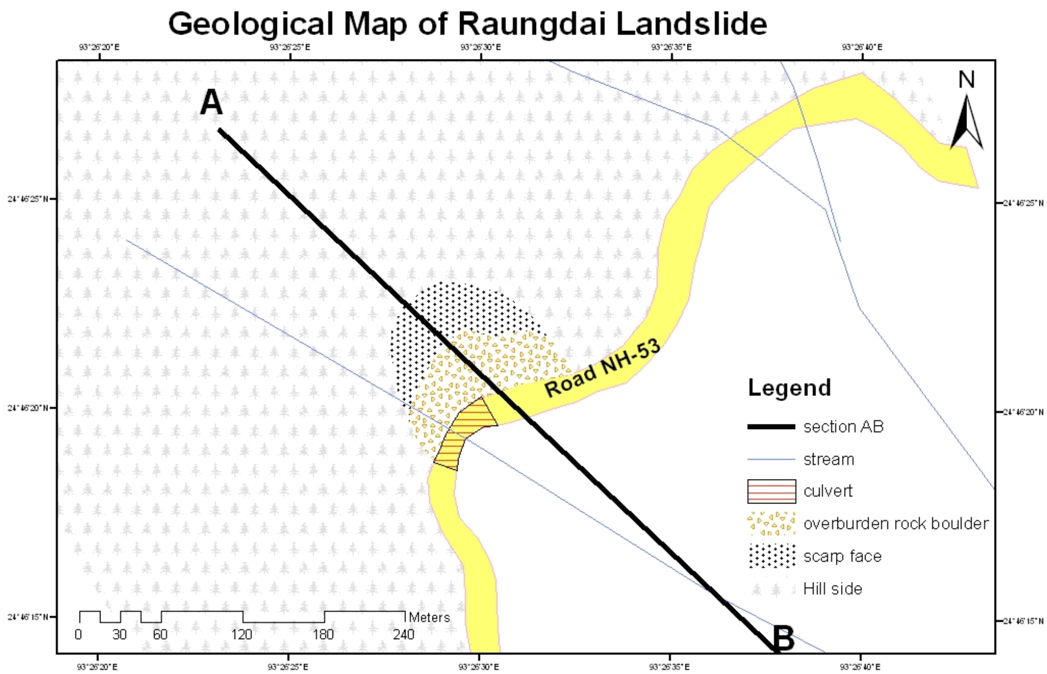


Fig 8: Geological Map of Raungdai Landslide

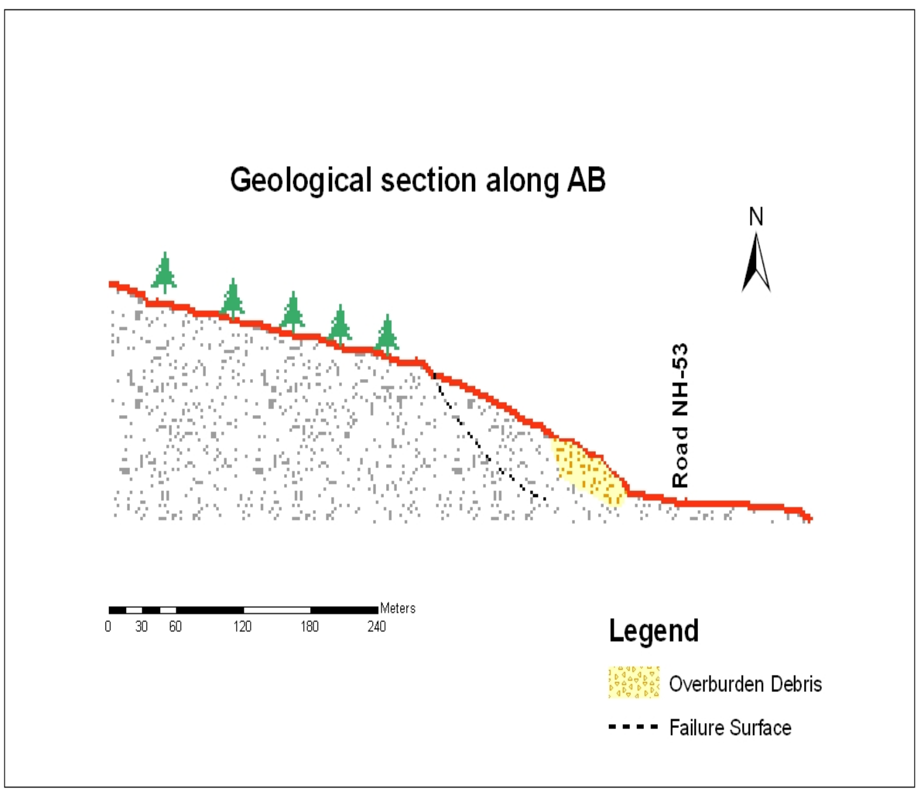


Fig 9: Geological along AB

18. ROCK MASS CLASSIFICATION OR ROCK MASS RATING

Bieniawski (1976) published the details of a rock mass classification called the Geomechanics Classification or the Rock Mass Rating (*RMR*) system. Over the years, this system has been successively refined as more case records have been examined and the reader should be aware that Bieniawski has made significant changes in the ratings assigned to different parameters. The discussion which follows is based upon the 1989 version of the classification (Bieniawski, 1989). Both this version and the 1976 version deal with estimating the strength of rock masses. The following five parameters are used to classify a rock mass using the *RMR* system:

1. Uniaxial compressive strength of rock material.
2. Rock Quality Designation (*RQD*).
3. Spacing of discontinuities.
4. Condition of discontinuities.
5. Groundwater conditions.

Table: 15 RMR Ratings (*after Bieniaswsky, 1989*)

<table>
<tr><td>PARAMETERS</td><td colspan="7">Range of Values</td></tr>
<tr><td rowspan="3">Point Load Index UCS (Mpa) Unconfined Compressive Strength</td><td>>10</td><td>10 – 4</td><td>4 – 2</td><td>2 – 1</td><td colspan="3"></td></tr>
<tr><td>>250</td><td>250 – 100</td><td>100 – 50</td><td>50 - 25</td><td>25-5</td><td>5-1</td><td><1</td></tr>
<tr><td>15</td><td>12</td><td>7</td><td>4</td><td>2</td><td>1</td><td>0</td></tr>
<tr><td rowspan="2">RQD (%) Rock Quality Designation</td><td>100 – 90</td><td>90 – 75</td><td>75 – 50</td><td>50 – 25</td><td colspan="3"><25</td></tr>
<tr><td>20</td><td>17</td><td>13</td><td>8</td><td colspan="3">5</td></tr>
<tr><td rowspan="2">Spacing of Discontinuties (mm)</td><td>>2000</td><td>2000-600</td><td>600-200</td><td>200-60</td><td colspan="3"><60</td></tr>
<tr><td>20</td><td>15</td><td>10</td><td>8</td><td colspan="3">5</td></tr>
<tr><td rowspan="2">Condition of Discontinuties</td><td>Extremely tight Very rough surfaces No separation Hard Joint wall rock</td><td>Very tight Slightly Rough surfaces separation <1mm Hard joint wall rock Not Continuous</td><td>Tight Slightly Rough separation <1mm No gouge Soft joint wall rock</td><td>Open slickensided walls or Gouge<5mm or Separation 1-5mm Continuous Joints</td><td colspan="3">Very Open Soft Gouge >5mm or separation >5mm Continuous Joints</td></tr>
<tr><td>30</td><td>25</td><td>20</td><td>10</td><td colspan="3">0</td></tr>
<tr><td rowspan="2">Groundwater in Joints (Pore pressure ratio)</td><td>COMPLETELY DRY</td><td>DAMP</td><td>WET</td><td>DRIPPING</td><td colspan="3">FLOWING</td></tr>
<tr><td>15</td><td>10</td><td>7</td><td>4</td><td colspan="3">0</td></tr>
</table>

Table: 16 Rock Mass Classes determined from Total Rating (after Bieniawski, 1989)

Rating	**100-81**	**80-61**	**60-41**	**40-21**	**<20**
CLASS	**I**	**II**	**III**	**IV**	**V**
Description	**Very good rock**	**Good rock**	**Fair rock**	**Poor rock**	**Very poor rock**

19. POINT LOAD TEST

The PLT is an attractive alternative to the UCS because it can provide similar data at a lower cost. The PLT has been used in geotechnical analysis for over thirty years (ISRM, 1985). The PLT involves the compressing of a rock sample between conical steel platens until failure occurs. The apparatus for this test consists of a rigid frame, two point load platens, a hydraulically activated ram with pressure gauge and a device for measuring the distance between the loading points. The pressure gauge should be of the type in which the failure pressure can be recorded. The International Society of Rock Mechanics (ISRM, 1985) has established the basic procedures for testing and calculation of the point load strength index. There are three basic types of point load tests: axial, diametral, and block or lump. The axial and diametral tests are conducted on rock core samples. In the axial test, the core is loaded parallel to the longitudinal axis of the core, and this test is most comparable to a UCS test. The point load test allows the determination of the uncorrected point load strength index (Is). It must be corrected to the standard equivalent diameter (De) of 50 mm. If the core being tested is "near" 50 mm in diameter (like NX core), the correction is not necessary. The procedure for size correction can be obtained graphically or mathematically as outlined by the ISRM procedures. The value for the Is50 is determined by the following equation.

$$\mathbf{Is50 = P/D^2}$$

P = Load at the Failure.
D =Diameter of the rock specimen.

As Hoek (1977) pointed out, the mechanics of the PLT actually causes the rock to fail in tension. The PLT's accuracy in predicting the UCS therefore depends on the ratio between the UCS and the tensile strength. For most brittle rocks, the ratio is approximately 10. For soft mudstones and claystones, however, the ratio may be closer to 5. This implies that PLT results might have to be interpreted differently for the weakest rocks. Early studies (Bieniawski, 1975; Broch and Franklin, 1972) were conducted on hard, strong rocks, and found that relationship between UCS and the point load strength could be expressed as:

$$\mathbf{UCS = (K)\ Is50 = 24\ Is50}$$

Where K is the "conversion factor." Broch and Franklin reported that for 50 mm diameter cores the uniaxial compressive strength is approximately equal to 24 times the point load index. They also developed a size correction chart so that core of various diameters could be used for strength determination. Subsequent studies found that K=24 was not as universal as had been hoped, and that instead there appeared to be a broad range of conversion factors.

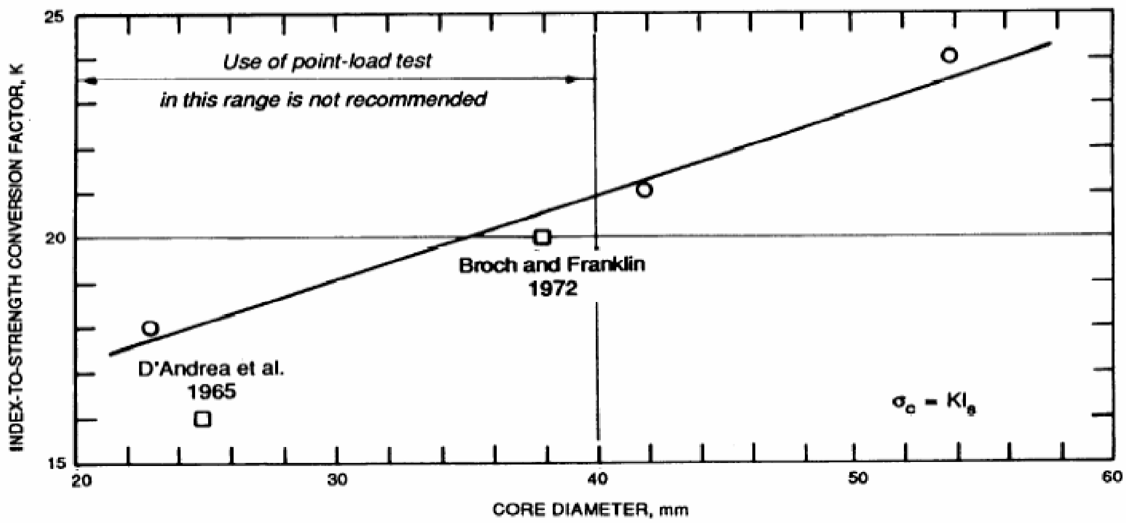


Fig 10: Size correlation graph for index-to-strength conversion (after Bienawski-1975)

The correlation of Is with UCS is both material specific and size dependent. Therefore, for best accuracy this correlation should be established for each site specific material. In this case, a number of UCS tests would be necessary, but the time and cost savings for large numbers of strength tests would be significant using the point load tester. On the average, UCS is 20-25 times the point load strength **(1SGO),** but can vary over a much wider range (ISRM 1985). Where site specific correlations or other material-specific information is not available, the UCS can be found using the size correlation graph (Figure 1) to obtain the index to-UCS conversion factors. For example**,** a conversion factor of 23 is found if using the common NX (54 mm) core size. Point load tests on igneous and the harder sedimentary rocks could be expected to have a reasonable correlation with UCS using factors close to those given above. However, the weaker rock materials, which are typically dredged by mechanical means, may require a lower correlation factor. The limited point load and UCS tests performed to date on weaker dredged material indicate an average correlation factor near 20. Correlation was certainly well within the variability of the material even using a factor of 23 for the (NX) sized core.

20. POINT LOAD STRENGTH

Determination of rock strength (Point Load Test)

Point load testing is used to determine rock strength indexes in geotechnical practice. The point load test apparatus and procedure enables economical testing of core or lump rock samples in either a field or laboratory setting. In order to estimate uniaxial compressive strength, index-to-strength conversion factors are used. More than 25 individual test results, from sliding site, were used in the study. Rock lithologies were classified into general categories and conversion factors were determined for each category. This allows for intact rock strength data to be made available through point load testing for numerical geotechnical analysis and empirical rock mass classification systems. The point load test (PLT) is an accepted rock mechanics testing procedure used for the calculation of a rock strength index. This index can be used to estimate other rock strength parameters. The focus of study is to correlate the point load test index (Is50) with the uniaxial compressive strength (UCS), and to propose appropriate Is50 to UCS conversion factors for different rock samples of the sliding site. The rock strength determined by the PLT, like the load frame strengths that they estimate, is an indication of intact rock strength and not necessarily the strength of the rock mass. The values obtained from point load strength of the study area are given in the table 17.

Table: 17 Point load testing result of the rock sample

Sample No	Thikness, D (cm)	Load, P (kn)	D2 cm2	Is = P/D2 (Mpa)
1	3.8	13.75	14.44	9.522160665
2	3.8	2.25	14.44	1.558171745
3	3.8	15.75	14.44	10.90720222
4	3.8	20.25	14.44	14.02354571
5	4.3	19.75	18.49	10.68144943
6	4.2	20.12	17.64	11.40589569
7	4.7	1.75	22.09	0.792213671
8	5	4.12	25	1.648
9	4.4	16	19.36	8.26446281
10	3.7	18.4	13.69	13.44046749
11	2.6	13.25	6.76	19.60059172
12	4.1	14.75	16.81	8.774538965
13	3.9	11	15.21	7.232084155
14	4.8	3	23.04	1.302083333
15	3.8	13.12	14.44	9.085872576
16	4.3	1.5	18.49	0.811249324
17	5.5	2.25	30.25	0.743801653

18	**4**	**9.5**	**16**	**5.9375**
19	**4.2**	**14.5**	**17.64**	**8.219954649**
20	**4.2**	**16**	**17.64**	**9.070294785**
21	**4.1**	**10.5**	**16.81**	**6.246281975**
22	**2**	**5.25**	**4**	**13.125**
23	**3.4**	**21**	**11.56**	**18.16608997**
24	**4.2**	**9**	**17.64**	**5.102040816**
25	**3.4**	**11.75**	**11.56**	**10.16435986**
Average Is = 8.23301253 Mpa				

21. ROCK QUALITY DESIGNATION INDEX (RQD)

The Rock Quality Designation Index (RQD) was developed by Deere (Deere et al 1967) to provide a quantitative estimate of rock mass quality from drill core logs. RQD is defined as the percentage of intact core pieces longer than 100 mm (4 inches) in the total length of core. The core should be at least NW size (54.7 mm or 2.15 inches in diameter) and should be drilled with a double-tube core barrel. Palmstrom (1982) suggested that, when no core is available but discontinuity traces are visible in surface exposures or exploration adits, the RQD may be estimated from the number of discontinuities per unit volume. The suggested relationship for clay-free rock masses is:

RQD = 115 - 3.3 Jv

Where Jv is the sum of the number of joints per unit length for all joint (discontinuity) sets known as the volumetric joint count. RQD is a directionally dependent parameter and its value may change significantly, depending upon the borehole orientation. The use of the volumetric joint count can be quite useful in reducing this directional dependence. RQD is intended to represent the rock mass quality in situ. When using diamond drill core, care must be taken to ensure that fractures, which have been caused by handling or the drilling process, are identified and ignored when determining the value of RQD. The rock quality designation (RQD) of the sliding site is given below:

RQD (%):-

$$\text{Rock Quality Designation, RQD} = 115 - 3.3 \times Jv$$
$$= 115 - 3.3 \times 6$$
$$= 95.2$$
$$where, Jv \text{ is the number of joints per cubic meter}$$

Table: 18 Rating of RQD in relation to fracture frequency *(Franklin et al, 1971)*

Term	RQD%	Fracture Frequency	Rating
Very Poor	0-25	>15	3
Poor	25-50	15-08	8
Fair	50-75	08-05	13
Good	75-90	05-01	17
Excellent	90-100	<01	20

22. DISCONTINUITIES PARAMETERS

22.1 Joint Spacing

Deere (1968) proposed the classification of joint spacing but several methods has also been used for direct discontinuity measurement. Joint spacing influences the permeability and seepage characteristics of the slope material. In the present study the rating scheme proposed by Bieniawski (1989) is used and the result obtained is shown in the table 21.

22.2 Conditions of Discontinuties

Conditions of Discontinuities include persistence of discontinuities, surface roughness, alteration and joint condition factor. The conditions of discontinuities is also taking important role in the rock instability.

22.2.1 Persistence or Continuity

The term persistence refers to the lateral extend of the discontinuity plane. Its size and length are functions of the thickness or separation of the discontinuity. The persistence determines the degree to which failure of intact rock would be involved in eventual failure (Deere et al., 1968). This rating scheme has been estimated after Bieniawski (1984) Palmstrom (1995) and observed in the sliding area as discontinuous and short & medium.

22.2.2 Surface Roughness

The inherent surface of smoothness, unevenness and waviness with respect to the discontinuity plane, is the surface roughness or deviation of a discontinuity surface from perfect planarity. An increase in the roughness of discontinuity planes results in the increased effective friction angle along the joint surface. Planar and smooth type of surface roughness is observed.

Table: 19 Roughness and Persistence Ratings (*After Palmstrom, 1995*)

Joint Persistence		Very Short	Short	Medium	Long/Large	Very Long
Length in meter		< 0.05	0.1 – 1.0	1 – 10	10 – 30	> 30
Rating	Continuous	3	2	1	0.75	0.5
	Discontinuous	6	4	2	1.5	1
Joint Roughness		Large Scale Waviness of Joint Plane				
	Small scale smoothness	Planar	Slightly Undulating	Strongly Undulating	Stepped	Interlocking
Rating	Very Rough	3	4	6	7.5	9
	Rough	2	3	4	5	6
	Slightly Rough	1.5	2	3	4	4.5
	Smooth	1	1.5	2	2.5	3
	Polished	0.75	1	1.5	2	2.5
	Slickensides	0.6 – 1.5	1 – 2	1.5 – 3	2 – 4	2.5 - 5

Remarks: *For Slickenside higher value is used; For Irregular & Filled Joints, jR=5 and jR=1 rating will be awarded respectively.*

22.2.3 Alteration

Alteration refers to the changes, which occur in the chemical and mineralogical composition of a rock, brought about by permeating hydrological fluids or by pneumatolytic action. The rating scheme of Alteration of the present study is adopted after Bieniawski (1973) and Palmstrom (1995) shown in the table below. Clean joints and Fresh rock wall is observed in the sliding area.

Table: 20 Rating of Discontinuities Alteration (after Palmstrom, 1995)

Contact between the two rock surface			
Term	**Sub - category**		**Rating**
Clean joint	Healed or welded joints		0.7
	Fresh rock wall		1.00
	Alteration of joint's wall	Less altered	2.00
		More altered	4.00
Coating or thin filling	Sand, silt, calcite etc.		3.00
	Clay, chlorite, talc etc		4.00

Filled joints, partly or no joint wall contact		
Type of filling material	**Rating**	**No wall contact, thick filling**
	Partly wall contact, thin filling (<5mm)	
Sand, silt, calcite etc	4	8
Compacted clay	6	10
Soft clay	8	12
Swelling clay	8 – 12	12 - 20

22.2.4 Joint Condition Factor

According to Palmstrom (1995), the joint condition factor can be calculated by using the following relations.

$$\text{Joint Condition Factor, jC} = \frac{jL \times jR}{jA}$$

$$= \frac{4+1}{1}$$

$$= 5$$

23. GROUNDWATER CONDITIONS

Groundwater is one of the most important parameters for assessment of the stability of rock slope. Important effect of the presence of groundwater in a rock mass is the reduction in the stability, resulting from water pressures within the discontinuities. Hydrogeological condition of the sliding area is influenced by a number of factors like climate, lithology, structural discontinuities, neotectonic activities, landuse and landcover, drainage pattern/network etc. Groundwater conditions is generally chanalised along structural discontinuity of rocks and observing the area as wet during the rainy season and damp during winter groundwater condition.

Table: 21 Discontinuities parameter of the sliding area.

Location	Frequency per meter	Spacing in meter	Persistence in meter	Surface roughness	Alteration	Groundwater
Raungdai / Blongdai 4 km to Nungba	6	0.16	0.1- 10 Discontinuous and short & medium	Planar and smooth	Clean joints and Fresh rock wall	Wet during summer & damp in winter

24. DETERMINATION OF RMR:

The numeric value of RMR is the algebraic sum of tabulated value calculated from field parameters. From the field data, strength of rock masses (RQD), spacing of joints, conditions of discontinuities and groundwater conditions for landslide location is calculated and their respective weightages have been assigned. The higher value of RMR indicates good quality of rock mass whereas lower the value of RMR indicates the very poor rock quality to enhance the possibility of sliding. The RMR value of the sliding area is tabulated in the table below.

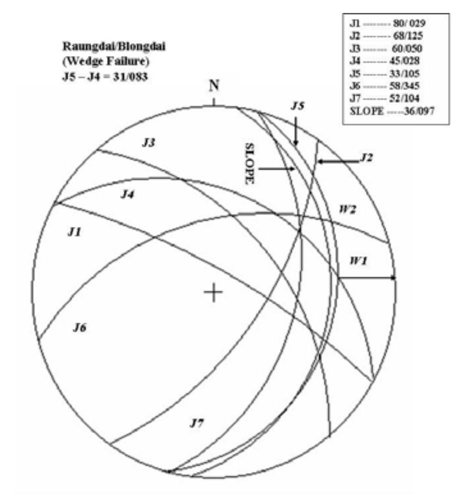
Table: 22 Determination of RMR

Location	Strength	RQD	Spacing	jL	jR	jA	jC	Ground Water	RMR
Raungdai / Blongdai 4 km to Nungba	**12**	**20**	**8**	**3**	**1**	**1**	**5**	**7**	**57 Fair rock**

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

25. KINEMATIC ANALYSIS THROUGH STEREOGRAPHIC PROJECTION

Modes of slope failure in jointed rock masses were examined kinematically using stereographic projection technique (Panet, 1969), which is purely geometric in nature. The angular relationships between discontinuities and slope surfaces are applied to determine the potential and modes of failure (Kliche, 1999). Hoek and Bray (2005), had established four modes of failures, viz, Planar, Wedge, Circular or Rotational and Toppling.



Stereo plot of discontinuities of Raungdai slide

Discontinuities data collected from the field are systematically processed and tabulated so that it can be effectively used to analyses stereographically using Georient – 9.4 software, to determine the modes of slope failures, shown in the table 23. The result obtained from stereo plots indicate that the landslide of the Raungdai is wedge failure.

Table: 23 Orientation of Discontinuities and slope

Station Name	aj	Bj	as	Bs	aj – as	Bj – Bs	Probable Failure
Raungdai	88	30	97	36	-9	-3	Wedge Failure

aj- Joint dip direction, **as**-Slope direction, **Bj**- joint dip, **Bs**- Slope angle

26. ESTIMATION OF SLOPE MASS RATING (SMR)

Table: 24 **SMR = RMR + (F_1 x F_2 x F_3) + F_4**

Adjusting Factors for Joints (F_1,F_2,F_3)	**aj = Dip Direction of Joint βj = Dip of Joint as = Dip Direction of Slope βs = Angle of Slope**				
	VERY FAVOURABLE	**FAVOURABLE**	**FAIR**	**UNFAVOURABLE**	**VERY UNFAVOURABLE**
Plane failure ׀ aj – as ׀	>30°	30° - 20°	20°- 10°	10°-5°	<5°
Toppling ׀ aj – as 180°׀ =	**0.15**	**0.40**	**0.70**	**0.85**	**1.00**
F1 Value	**F_1 = (1 – sin ׀ aj – as ׀)²**				
׀ βj׀ = F2 value	<20°	20° - 30°	30°- 35°	35°- 45°	>45°
Plane	**0.15**	**0.40**	**0.70**	**0.85**	**1.00**
Toppling	**1.00**				
	F_2 = tan2 βj				
Plane Failure βj- βs	>10°	10° - 0°	0°	0° - (-10°)	<(-10°)
Toppling βj+ βs	<110°	110° - 120°	>120°	-	-
	0	**-6**	**-25**	**-50**	**-60**
F_3 Value	F_3 (Bieniawski adjustment Ratings for Joints Orientation, 1976)				
F_4 Adjusting Factor for Excavation Method	F_4 = Empirical values for method of Excavation`				
	Natural Slope	Pre Splitting	Smooth Blasting	Blasting or Mechanical	Deficient Blasting
F_4 Value	**15**	**10**	**8**	**0**	**-8**

Table: 25 SMR Classes (after Romana, 1985)

Class	**Vb**	**Va**	**IVb**	**IVa**	**IIIb**	**IIIa**	**IIb**	**IIa**	**Ib**	**Ia**
	1-10	**11-20**	**21-30**	**31-40**	**41-50**	**51-60**	**61-70**	**71-80**	**81-90**	**91-100**
DESCRIPTION	**VERY BAD**		**BAD**		**FAIR**		**GOOD**		**VERY GOOD**	
STABILITY	**COMPLETELY UNSTABLE**		**UNSTABLE**		**PARTIALLY STABLE**		**STABLE**		**COMPLETELY STABLE**	
FAILURE	**BIG PLANAR or SOIL LIKE**		**PLANAR or BIG WEDGES**		**SOME JOINTS or MANY WEDGES**		**SOME BLOCKS**		**NONE**	
SUPPORT	**REEXCAVATION**		**IMPORTANT/CORRECTIVE**		**SYSTEMATIC**		**OCCASIONAL**		**NONE**	

The adjustment factor **F_1, F_2, F_3** for discontinuities can be determined from the values of orientation of discontinuities and slope obtained from stereo-plot of the landslides sites. The probable mode of failure can also be observed from the stereo-plots. The result obtained from stereo plots indicate that the landslide of the Raungdai is wedge failure. In case of wedge failure two discontinuities strike obliquely across the slope face with their line of intersection getting daylighted on the line of intersection, provided that the plunge of line of intersection is less than the slope inclination and greater than the angle of internal friction. The value of F4 is taken with reference to the mode of excavation available in the sliding site. Thus, the SMR for Raungdai landslide site has been estimated by calculating the formula i.e. SMR = RMR + (F_1 x F_2 x F_3) + F_4 and probable stability of the area is shown in the table 26.

Table: 26 Estimation of SMR of the Raungdai slide

Station Name	RMR	F_1	F_2	F_3	F_4	SMR	Description	Stability
Raungdai / Blongdai	**57**	**1**	**0.4**	**-50**	**0**	**37**	**BAD**	**UNSTABLE**

27. FACTOR RESPONSIBLE FOR SLOPE FAILURE

Detailed studies of the above two slides have found that both Natural and Anthropogenic causes are main responsible factors for the slope failures.

27.1 Natural Causes:

a) Whole area falls under young Tertiary sediments, which can not sustain the natural processes of weathering and erosion. Presence of clay and shale has also aggravated the slope failure when they become wet.

b) Geological structure and tectonic activity have an inherent in the sites under investigation. Stresses created folding, faulting have resulted in jointing, shearing and imbrication structures.

c) High relief and steep slope prevailing in these two site are not conducive for the stability of slope.

d) The region receives highest rainfall in the state. Therefore heavy precipitation has led to oversaturation, causing loss in shearing resistance with the increment in weight. Surface runoff induces rapid weathering and erosion of the slope materials, decreasing the angle of repose.

27.2 Anthropogenic Causes:

a) Deforestation for domestic purposes, current jhuming practices on the slopes in the proximity of the highway and catchment area lead to slope failure.

b) Improper land-use and land-cover practices are the excessive hill slope cutting either due to quarrying, construction of new road and widening of the existing roads has led to instability of slope.

c) In the tectonic factors, excessive quarrying for construction materials irrespective of the stability of the slope has led to such small scale sliding along the highway.

28. PREVENTIVE/MITIGATIVE MEASURES

Complete prevention of landslide is a very difficult task. However, we can minimize the effects of landslides especially the smaller ones and those provoked by human activities. Because vulnerability to landslide hazard is a function of location, type of human activity, use and frequency of landslide events. In a generalized manner, we can observe three important steps to minimize the landslide hazards.

- We should avoid construction in areas, which are prone to slide and subsidence.
- Where slopes are naturally stable, we should construct in such a way that does not make them unstable.

- Saturation of ground material is a critical factor of landslide, so we must develop good and effective drainage system so that slope materials do not become water logged and likely to slide or flow.

28.1 Environmental/Anthropogenic Mitigative Measures

Deforestation and landuse are the principal causes of landslide aggravation. In areas along NH 53 where there is landslide, excessive landuse is also a principal factor. Landslide and subsidence are also common in areas where terrace cultivation is practiced. So the following remedial measures may be taken up.

- Upper reaches of the slope(s) should be used mainly for plantation of horticultural plants, cash crops and others where retaining of water is not required.
- Terrace cultivation must be practiced only in lower reaches of slopes, and in relatively gentle slopes of river/stream banks.
- Aforestation and mixed cropping of slopes may be encouraged, but plantation of big trees especially in moderate and high slopes should be avoided.

29. RESULTS AND DISCUSSION

The present analysis gives that the study area has been categorised into three different hazard zones i.e. Very High Hazard, High Hazard and Moderate Hazard zones. Majority of the study area falls in the High Hazard and Very High Hazard zone. High Hazard Zone is about 60.58% of the total area covering 104.8 sq.km followed by Very High Hazard Zone covering 40.2 sq.km with 23.24% of the total area. 25.2 sq. km. and 14.57% area is included in Moderate Hazard Zone, whereas Very Low Hazard Zone (VLH) and Low Hazard Zone (LH) are not present.

The field validation of High hazard and Very High hazard zones is done and the LHZ methodology for the present study is found very useful. Eighteen (18) incidences of landslides have been found in present study. Out of these, ten number of landslides fall in the High Hazard Zones which is about 55.5% of the total slides and eight number of landslides are occurred in the Very High Zones which is 44.5% of the total slides. There is no incidence of landslide so far in the Moderate Hazard Zones. The remedial measures are attempted by the Border Road Organisation, however, the frequency of landslide triggering is still quite high. The various factors that trigger the landslides in the study area are mainly from the typical geological formation of Tertiary Group that are highly fractured, jointed and prone to weathering. During the heavy monsoon, soft lithology like shale and mudstone becomes mud and silt and susceptible to sliding. The slope of the terrain is 30-40 degree under HH and

VHH zones which is favorable for sliding of debris. Another factor worth mentioning is the cash crop cultivation along the roadside which also contributes to the landslide triggering phenomenon. The scope of the future study work is to undertake large scale geotechnical solution of vulnerable slides in the study area.

30. CONCLUSIONS

The present studies highlight the application of Remote Sensing techniques and GIS in preparation of landslide hazard zonation mapping along NH-53 in Manipur. Results from the studies highlight the Very High and High landslide hazard zones in the study area which is validated by 18 incidences of active and old landslides. The landslide triggering phenomenon are moderate to steep slope of terrain, cash crop cultivation practices along the road side and heavy rainfall. The GIS data base of the landslide hazards for the study area may be used for future detailed geotechnical solutions to stabilise the landslides.

31. PHOTOGRAPHS OF LANDSLIDE ACTIVATED AREA

Landslide at Nungba

Landslide at Raungdai

Construction of retaining wall to avoid Landslide at Awangkhul Pt. II near Army Outpost

Collapse of the same Retaining wall observed

Landslide near Rongkhong Bridge

Old Slide at Awangkhul

Landslide at Awangkhul Pt. II (Ragailong)

Landslide at Khumji near Noney


Landslide at Awangkhul Pt. II near Army Outpost


Landslide at Awangkhul Pt. II near Army Outpost


Landslide at Awangkhul Pt. II (Ragailong)


Landslide at Awangkhul Pt. II (Ragailong)


Landslide at Awangkhul Pt. II (Ragailong)


Landslide at Khumji near Noney


Landslide at Awangkhul Pt. II near Army Outpost


Landslide at Khumji near Noney

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