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THESIS NO. PUL079MSGtE003

**Stability Analysis and Mitigation Design of Excavated Slope Failure: A Case**  
**Study of Tuin Khola Landslide**

by

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A THESIS

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DEGREE OF MASTER OF SCIENCE IN  
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DEPARTMENT OF CIVIL ENGINEERING  
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## DECLARATION

I, Aayush Yadav, hereby declare that the thesis entitled “Stability Analysis and Mitigation Design of Excavated Slope Failure: A Case Study of Tuin Khola Landslide” is my original work prepared for the partial fulfillment of the requirements for the degree of master of science in Geotechnical Engineering under department of civil engineering at Pulchowk Campus under Tribhuvan University.

This work has not been submitted to any institution for any kind of degree. The information derived from published work has been cited in the text and included in the reference.

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## **ABSTRACT**

The failures in the Narayanghat-Mugling (NM) road happen all the time. This study investigates a slope instability at Tuin Khola along the NM Road section. A multi-disciplinary approach was employed, including borehole drilling, geological mapping, and Rock Mass Rating (RMR). Subsurface weaknesses were identified through geophysical methods. Stability was modeled using SLOPE/W via Limit Equilibrium Method (LEM) across dry, monsoon, and seismic scenarios, with results validated using Phase2 Finite Element analysis.

Numerical modeling identified critical instability sections with a minimum Factor of Safety (FS) of 0.471 at CH 00+100 during rainfall and a maximum FS of 2.718 at CH 00+020 under dry conditions. Analysis confirmed that failure surfaces are primarily situated at the colluvium-bedrock interface, where monsoon-induced pore water pressure significantly degrades stability.

To protect the road and surrounding area, a more efficient stabilization plan was evaluated. Results demonstrate that implementing 25 mm diameter rock bolts (5–10 m length, 1.5 m x 2 m spacing) alongside drainage enhancement and RCC cladding successfully raises the FS to meet safety thresholds (>1.2 for rainfall; >1.1 for seismic). This provides a technically sound and cost-effective framework for slope management in the Lesser Himalayan region.

**Keywords:** GeoStudio; Monsoon; Stability; Slopes; Threshold

# TABLE OF CONTENTS

COPYRIGHT.....	i
DECLARATION.....	iii
ACKNOWLEDGEMENTS.....	iv
ABSTRACT.....	v
TABLE OF CONTENTS.....	vi
LIST OF TABLES .....	x
LIST OF FIGURES.....	xii
ABBREVIATIONS AND ACRONYMS .....	xiv
1. INTRODUCTION .....	1
1.1. Background .....	1
1.2. Location of Study Area .....	3
1.3. Statement of the Problem .....	4
1.4. Research Questions .....	5
1.5. Research Objectives .....	5
1.6. Significance of the Study .....	6
1.7. Scope of the Study .....	6
1.8. Organization of the Thesis .....	7
2. LITERATURE REVIEW .....	8
2.1 Landslide Investigations in Mountainous Areas.....	8
2.2 Important Geotechnical Factors for Analyzing Slope Stability.....	8
2.3 Landslide Susceptibility Analysis along the NM Highway .....	8

2.4	Landslide in the Lesser Himalayan Region of Nepal .....	9
2.5	Slope Hazards and Infrastructure Vulnerability along NM Highway .....	9
2.6	Numerical Modeling for Evaluating Slope Stability .....	9
2.7	Determination of $c$ and $\phi$ from RocLab .....	10
2.8	Soil Nailing Technique: A Case Study of NM Road Section .....	10
2.9	Stability Assessment of Landslides of Selected Stretches of NM Road .....	10
2.10	Climatic Conditions .....	11
2.11	Slope Stability Analysis .....	12
2.11.1	Plate tectonic concept .....	12
2.11.2	Seismicity .....	13
2.12	Numerical Analysis .....	14
2.12.1	General procedure for slope stability analysis in slope/w .....	14
2.13	Rock Mass Classification .....	15
3.	STUDY AREA AND DATA .....	20
3.1	General Geology, Geomorphology .....	20
3.2	Slope Characteristics .....	22
4.	METHODOLOGY .....	23
4.1	Inspection Phase .....	23
4.1.1	Site reconnaissance survey .....	23
4.1.2	Field observations .....	24
4.1.3	Preliminary on site data collection .....	24
4.1.4	Geological mapping .....	24
4.1.5	Location Logging and sampling .....	25
4.1.6	Rock mass characterization .....	25
4.1.7	Ground water table monitoring .....	25

4.1.8	Hydrological study.....	26
4.2	Investigation Phase .....	26
4.2.1	Geophysical study.....	26
4.2.2	Geological assessment of slopes.....	27
4.2.3	Hydrological assessment.....	36
4.2.4	Geotechnical study.....	37
4.3	Analysis Phase .....	39
4.3.1	Detailed study .....	39
4.3.2	Profile characterization .....	40
4.3.3	Slope stability assessment.....	40
4.3.4	Final recommendation .....	42
5.	RESULTS AND DISCUSSION.....	43
5.1	Main Causes for Occurrence of Slide.....	43
5.2	Geotechnical and Geophysical Analysis.....	43
5.2.1	Interpretation of geotechnical data & findings of core drillings.....	45
5.3	Geophysical Investigation.....	46
5.3.1	Interpretation of geophysical data.....	47
5.4	Slope Stability Assessment .....	48
5.5	Sub-Surface Conditions .....	48
5.6	Material Properties.....	48
5.7	Hydraulic Properties .....	50
5.8	Seismic Coefficient.....	50
5.9	Slope Stability Analysis .....	51
5.10	Kinematic Analysis Result.....	54
5.11	Numerical Modelling with Slope/w .....	55

5.12	Groundwater and Stability Conditions.....	56
5.13	Results of Slope Stability Analysis.....	57
5.14	Analysis Results Validation with Phase2 (FEM).....	59
5.15	Protection Design Drawing of Slope Failure.....	60
5.16	Results Verification.....	63
6.	CONCLUSIONS AND RECOMMENDATIONS .....	64
6.1	Conclusions .....	64
6.2	Recommendations.....	65
	REFERENCES .....	66
	ANNEX A: Geophysical, Geological, Geotechnical Analysis Data. ....	69
	ANNEX B: Photographs .....	87
	ANNEX C: Publication .....	91
	ANNEX D: Similarity Report .....	100

## LIST OF TABLES

<i>Table 1-1: Summary of major work</i> .....	7
<i>Table 2-1: General procedure for slope stability analysis in Slope/w</i> .....	14
<i>Table 2-2: Rock mass class based on RMR</i> .....	16
<i>Table 2-3: Q- system parameters</i> .....	16
<i>Table 2-4 Rock mass classification based on Q- system</i> .....	17
<i>Table 4-1: Rock mass data collection on site</i> .....	25
<i>Table 4-2: Works RMR calculation (Bieniawski 1989) at L1</i> .....	27
<i>Table 4-3: RMR calculation of failure slope L1</i> .....	28
<i>Table 4-4: Joint sets and mean joint from random joints at L1</i> .....	28
<i>Table 4-5: Works RMR calculation (Bieniawski 1989) at L2</i> .....	29
<i>Table 4-6: RMR calculation of failure slope L2</i> .....	30
<i>Table 4-7: Joint sets and mean joint from random joints at L2</i> .....	30
<i>Table 4-8: Works RMR calculation (Bieniawski 1989) at L3</i> .....	31
<i>Table 4-9: RMR calculation of failure slope at L3</i> .....	32
<i>Table 4-10: Joint sets and mean joint from random joints at L3</i> .....	32
<i>Table 4-11: Works RMR calculation (Bieniawski 1989) at L4</i> .....	33
<i>Table 4-12: RMR calculation of failure slope at L4</i> .....	34
<i>Table 4-13: Joint sets and mean joint from random joint at L4</i> .....	34
<i>Table 4-14: Works RMR calculation (Bieniawski 1989) at L5</i> .....	35
<i>Table 4-15: RMR calculation of failure slope at L5</i> .....	36
<i>Table 4-16: Joint sets and mean joints from random joints at L5</i> .....	36
<i>Table 5-1: Lab results: colluvial debris &amp; rock block</i> .....	45

<i>Table 5-2: Laboratory test data of core and rock block sample .....</i>	<i>45</i>
<i>Table 5-3: Interpretation of MAM of Tuin khola, landslide .....</i>	<i>46</i>
<i>Table 5-4: Estimation of geo-mechanical properties of bedrock.....</i>	<i>49</i>
<i>Table 5-5: Stereographic projection of rock mass exposed at slide area .....</i>	<i>54</i>
<i>Table 5-6: FS comparison with case .....</i>	<i>56</i>
<i>Table 5-7: Results of slope stability analysis.....</i>	<i>57</i>

## LIST OF FIGURES

<i>Figure 1-1: Aerial view of after landslide after landslide.</i> .....	2
<i>Figure 1-2: Slope characteristic of the proposed study area.</i> .....	2
<i>Figure 1-3: failure plane and geometry of the landslide.</i> .....	3
<i>Figure 1-4: Aerial view of after landslide after landslide.</i> .....	4
<i>Figure 1-5: View of slope material of roadside cut slope.</i> .....	4
<i>Figure 2-1: Geographical map of the study area (Source: Devkota et al. 2013).</i> .....	11
<i>Figure 2-2: Major tectonic boundary and various longitudinal zones.</i> .....	12
<i>Figure 2-3: Seismic hazard map of Nepal (source: USGS).</i> .....	13
<i>Figure 2-4: Historical events of earthquakes in Nepal.</i> .....	14
<i>Figure 2-5: GSI for jointed rocks.</i> .....	18
<i>Figure 3-1: Tuin khola area (CH 33+900), before massive landslide.</i> .....	20
<i>Figure 3-2: Geological map of Nepal with the study area.</i> .....	21
<i>Figure 3-3: Geological setting NM region (Source: DMG, 2020).</i> .....	21
<i>Figure 3-4: Digital elevation model of tuin khola area.</i> .....	22
<i>Figure 4-1: Overall methodology of the study.</i> .....	23
<i>Figure 4-2: Dispersion curve of MAM.</i> .....	27
<i>Figure 4-3: Exposure of rock mass at L1.</i> .....	27
<i>Figure 4-4: Exposure of rock mass at L2.</i> .....	29
<i>Figure 4-5: Exposure of rock mass at L3.</i> .....	31
<i>Figure 4-6: Exposure of rock mass at L4.</i> .....	33
<i>Figure 4-7: Exposure of rock mass at L5.</i> .....	35
<i>Figure 4-8: Core samples recovered at Tuin khola.</i> .....	37

<i>Figure 4-9: Debris samples for sieve analysis.</i> .....	38
<i>Figure 4-10: UCS test of rock sample.</i> .....	39
<i>Figure 4-11: Slope stability model in Geo-studio at locations (L3).</i> .....	41
<i>Figure 5-1: Borehole log sheet of horizontal drilling.</i> .....	44
<i>Figure 5-2: Shear wave velocity MAM test.</i> .....	47
<i>Figure 5-3: Estimation of geo-mechanical properties of bedrock.</i> .....	50
<i>Figure 5-4: Probabilistic seismic hazard of Nepal (Source: DMG, 2002).</i> .....	51
<i>Figure 5-5: Contour map drawing of failure slope.</i> .....	52
<i>Figure 5-6: Differential stress distribution and displacement vector.</i> .....	53
<i>Figure 5-7: Stability analysis for the dry condition.</i> .....	55
<i>Figure 5-8: Stability analysis with fully saturated soil.</i> .....	55
<i>Figure 5-9: Stability analysis with fully saturated soil with seismic loading.</i> .....	56
<i>Figure 5-10: Circular failure analysis: Soil &amp; weathered rock (Ch: 0+060).</i> .....	57
<i>Figure 5-11: Comparison of result L3 for dry, saturated and seismic loading.</i> .....	59
<i>Figure 5-12: Protection of slope at Ch.: 00+020.</i> .....	60
<i>Figure 5-13: Protection of slope at ch: 00+040.</i> .....	61
<i>Figure 5-14: Protection of slope at Ch.: 00+060.</i> .....	61
<i>Figure 5-15: Protection of slope at Ch.: 00+100.</i> .....	62
<i>Figure 5-16: Protection of slope at Ch.: 00+120.</i> .....	62
<i>Figure 6-1: Layout of mitigate measures are recommended.</i> .....	65

## ABBREVIATIONS AND ACRONYMS

D	Disturbance Factor
DEM	Digital Elevation Model
DMG	Department of Mines and Geology
ERT	Electrical Resistivity Tunnel
FEM	Finite Element Model
GPS	Global Positioning System
GRC	Ground Reaction Curve
GSI	Geological Strength Index
ISMB	Indian Standard Medium Beam
H/D	Height/Diameter
IS	Indian Standard
LDP	Longitudinal Deformation Profile
MAM	Microtremor Array Measurement
NM	Narayanghat-Mugling
RMR	Rock Mass Rating
RQD	Rock Quantity Designation
RS	Rocscience
SCC	Support Characteristic Curve
SMR	Slope Mass Rating

# 1. INTRODUCTION

## 1.1. Background

Road network development forms important infrastructure for social and economic activities for all the countries in the world. During the infrastructure development roadside slope instability is a worldwide geo- hazard associated problem induced by harsh natural situation such as steep, unstable terrain, heavy rain, and seismic activity pose constant risks to landslides, slope failures, rock falls, rock mass failures, debris flow, and toe cutting especially in hilly roads and riverbank roads. The road slope disaster management plays key role in delivering efficient and effective transport services to road users (Meimei et al. 2026).

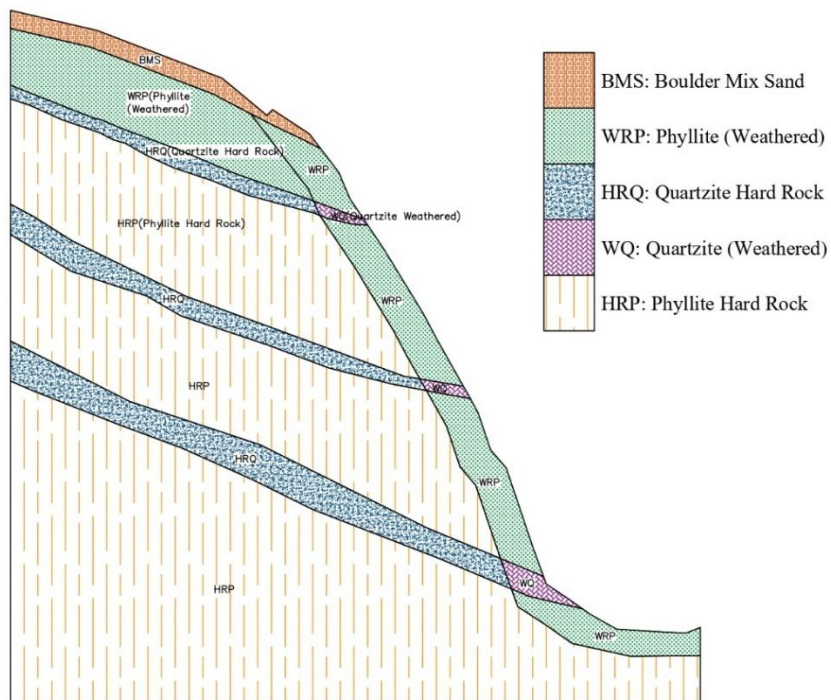
Since Nepal is a mountainous country, most of its road networks lie in the hilly region, and these roads have been experiencing major instability problems. The case of Kantilokmarga, Dharan-Dhankuta, Banepa-Bardiwas, Lamosangu-Jiri, Krishnabhir road sections are some examples that have slope instability problems. In addition, many road slope instabilities are presently observed, highlighting the need for awareness of geotechnical and environmental aspects in management. Addressing the NM slope instability problems at various sections requires detailed geotechnical studies for long-term mitigation and full highway operation. Landslides are one of the most destructive natural hazards in hilly regions (Mandip et al. 2025; Vittoria et al. 2025). The NM Highway is a vital lifeline for Nepal, serving as the primary corridor for trade, public transit, and military logistics while providing access to sacred religious sites (Kelechi et al. 2025).

Preliminary evidence points to a combination of human intervention and natural instability as the likely drivers of the failure (Liang et al. 2026). This study aims to isolate these primary failure drivers, providing the empirical data necessary to develop an effective, evidence-based stabilization strategy. Therefore, a comprehensive scientific investigation is essential to accurately identify the primary factors influencing this slope instability and provide the cost effective mitigation measures.



*Figure 1-1: Aerial view of after landslide after landslide.*

The specific landslide visible in the figure is a form of rock slope failure often exacerbated by the road-widening cuts that expose these unstable geological structures.



*Figure 1-2: Slope characteristic of the proposed study area.*

The Tuin Khola bridge area is known for its susceptibility to frequent rainfall-induced landslides, a result of the fragile geology of the Himalayas, which has been worsened by infrastructure development such as road construction. Local terrain comprises highly-jointed, fractured, and weak metamorphic rocks such as schist, phyllite, and quartzite. During heavy monsoon rainfall, these formations get saturated, which increases pore-water pressure in them and drastically reduces their stability (Xudong et al. 2025). Construction efforts for the NM road involved cutting into such steep and unstable hillsides. The current 60-meter arch bridge opened traffic in late 2024 or early 2025 and replaced an earlier one in a spot where widening of the road has created a new, expansive zone susceptible to landslide failure, now originating higher up the slope. The map shows the landslide area with the existing road (Figure 1-1), RCC cladding wall previously constructed. The rock outcrop and colluvium deposits were observed in all of slide's upslope to the road.

## 1.2. Location of Study Area

The study place is located at 1 km from Mugling Bazar towards the southern side near Tuin khola, NM road section. The co-ordinates of this particular area is 27.842978, 84.558697. This aerial view site is shown in figure 1-3.



*Figure 1-3: failure plane and geometry of the landslide.*



*Figure 1-4: Aerial view of after landslide after landslide.*

### **1.3. Statement of the Problem**

A landslide has occurred along Tuin khola, NM Road, indicating potential slope instability arising from both anthropogenic and natural causes. Preliminary observations suggest that toe-cutting for road expansion, unfavorable geological conditions, and light rainfall may have jointly contributed to the slope failure.



*Figure 1-5: View of slope material of roadside cut slope.*

The widening of the NM road, has severely destabilized naturally fragile, weak, and highly fractured hillsides. The lower section of the slope comprises competent rock beds, while the upper section consists primarily of colluvial soil deposits, which are loose and cohesion less. It is highly probable that the movement was caused by a combination of the slope's heterogeneous material, the erosion of toe support, and high pore-water pressure from rainfall. Given these complexities, a detailed site investigation is essential to pinpoint the primary drivers of this instability. The complex terrain highlights the need for innovative design and robust mitigation strategies to sustain the highway's functionality and ensure safe travel for its users.

#### **1.4. Research Questions**

Building on the challenge of understanding slope failure in such a critical transport corridor, this study focuses on evaluating stability through detailed back analysis of the Tuin Khola section. Specifically, this research seeks to address the following:

- How do the engineering properties of soil and debris vary across different slope segments (crown, middle, toe), and how do these variations influence slope stability?
- What is the Safety factor of the slope under different scenarios (natural, wet/rainy season, earthquake loading, and with reinforcement)?
- Which integrated stabilization measures including drainage, structural support (e.g., rock bolts/retaining structures), and bio-engineering?
- How accurate and reliable is the numerical model when validated against IS code recommendations and observed field behavior?

#### **1.5. Research Objectives**

The primary objective of this study is to analyze the unstable Tuin Khola section of the Narayanghat–Mugling Road and to evaluate appropriate mitigation measures for slope stabilization. The specific objectives are as follows:

- To evaluate the topographic, geological, geotechnical, and hydrological conditions of the slope section.
- To identify potential failure mechanisms using numerical analysis.

- To propose suitable mitigation and stabilization measures to reduce slope failure risk and improve overall stability.

### **1.6. Significance of the Study**

The continuous landslide observed in this area. Every monsoon season, the slopes around Tuin Khola become dangerously unstable, leading to frequent landslides that sever this essential connection. These recurring disruptions don't just cause traffic headaches, they have a real, lasting impact on the country's economy and the daily lives of those who rely on this route for transit and commerce. Frequent rainfall-induced landslides have disrupted transportation, posing significant risks to road users and causing recurring traffic blockages. Although annual clearance and short-term repair works are carried out to enable temporary mobility, these measures have proven insufficient, resulting in cumulative delays and increasing economic losses. Therefore, a detailed study and long-term mitigation strategy for landslide/slope failure management in the Tuin khola NM section is critically necessary to ensure continuous and safe operation of the highway.

### **1.7. Scope of the Study**

- To operate drone and develop images of the study area
- To carry out detailed topographical survey in 1m contour interval and record all the natural and existing features in the study area
- To carry out Geophysical Investigations Microtremor Array Measurement (MAM) along and across the slope within the study area
- To carry out horizontal drilling works to substantiate results of the Geophysical investigations
- To carry out necessary field tests and laboratory tests.
- To carry out slope stability analysis and analyze the existing slope for the selection and design of suitable mitigation measures

## 1.8. Organization of the Thesis

The thesis consists of chapter 1-5 with certain objectives which is explained below:

Chapter 1 contains brief description of slope stability followed by statement of problem of the Tuin khola, NM road section, research objectives, and significance of study, scope of the study.

Chapter 2 contains literature review which includes paper and journal review, methods used in surface settlement analysis, description about various models used during numerical analysis and rock mass classification system.

Chapter 3 contains research methodology which contains research design and research approach of the thesis.

Chapter 4 includes numerical analysis and results which contains detailed analysis result and comparison with input parameter. And result validation of the numerical value obtained from various software with field and codes.

Chapter 5 includes conclusion obtained from the study and recommendation for further study.

*Table 1-1: Summary of major work*

S. No.	Description	work
1	Inspection phase	Field visit, preliminary survey, data collection
2	Investigation phase	Geophysical test, geotechnical analysis
3	Analysis phase	Profile characterization, slope stability assessment

## **2. LITERATURE REVIEW**

### **2.1 Landslide Investigations in Mountainous Areas**

Landslides are a common geohazard in mountainous areas, especially in places like the Himalayas, where they cause a lot of damage to infrastructure, cost a lot of money, and put people's lives in danger (Sandipan 2025; Sreedeeep 2025). It is very important to come up with good ways to deal with these kinds of events, and this depends a lot on accurate hazard mapping, thorough assessment, and the right geotechnical treatments (Nauman et al. 2026). To fully understand how landslides happen and to come up with good ways to stop or control them, it is very important to do a detailed slope stability analysis (Adamu et al. 2023; Haixing et al. 2025). This analysis generally entails pinpointing the precise geotechnical and geological factors contributing to instability, in conjunction with assessing the engineering characteristics of the soil and rock materials located in various sections of the slope. Proposed Research on Landslide Assessment at Tuin Khola, NM Road.

### **2.2 Important Geotechnical Factors for Analyzing Slope Stability**

An accurate slope stability analysis, with a constant geotechnical parameters. These factors include the same shape of the slope, the possible depth of the sliding surface, and the level of Ground Water Table (Gwt), the unit weight of the soil and rock materials that are acting on the slope. Their shear strength properties of the soil and rock are very important (Haixing et al. 2025). These essential parameters constitute the failure slope for any predictive landslide model (Zhice et al. 2024). For example, the "Soil Investigation of Tuin Khola, NM Road" has generalize hoek and brown classification that show the specific rock design values.

### **2.3 Landslide Susceptibility Analysis along the NM Highway**

Landslide susceptibility map uses an essential tool for managing a vital transport corridor frequently disrupted by monsoon-triggered landslides. Recent research has shown that we can effectively identify high-risk zones by leveraging accessible geospatial data specifically FAO soil terrain maps and Google Earth Pro imagery, processed within ArcMap 10.8 (Prince et al. 2024). Using several parameters, the study reported that most sections of NM fall under high to very high susceptibility zones, and more than 18 vulnerable points were identified requiring priority mitigation. These findings mirror established trends in Himalayan geological research, confirming that topography, local geology, and monsoon rainfall are the

primary drivers of slope instability along our transport corridors (Nimai and Shraban 2024; Prince et al. 2024). By integrating these datasets, we can shift from reactive maintenance to proactive hazard management, ensuring a safer and more resilient transportation network across Nepal's mountainous landscape.

#### **2.4 Landslide in the Lesser Himalayan Region of Nepal**

The Tuin Khola study offers critical insights but operates within specific parameters. The reliance on limited subsurface data and short-term groundwater observations means that long-term, seasonal, and spatial variations are not fully accounted for. Additionally, using laboratory-derived parameters for modeling introduces potential uncertainties stemming from sample heterogeneity. Future studies could significantly strengthen this work by incorporating seismic load analysis and long-term displacement monitoring to better map the slope's evolution and reactivation risks.

#### **2.5 Slope Hazards and Infrastructure Vulnerability along NM Highway**

Numerical analysis has developed into a dependable and highly efficient technique for examining landslide behavior in future (Vito et al. 2025; Yan et al. 2022). These advanced techniques can be generally classified into LEM and finite element method (Bohao et al. 2025). Numerical models can show its deformation over time when different loads are apply on it (Kamran and Liusheng 2025). This modelling makes it possible to accurately assess slope instability and the risks that come with it. The Finite Element Method is best for looking at complicated shapes and rock masses that are not uniform.

#### **2.6 Numerical Modeling for Evaluating Slope Stability**

This evidence-based way to handle landslide risks. By combining a detailed slope inventory with smart hazard mapping. The methodology is well-aligned with international standards, particularly in how it filters out unnecessary alarms in already-stable areas.

If we build on this by incorporating modern protection techniques like advanced mesh and barrier systems, it can make this infrastructure much more resilient. The Nepal has prioritize critical sections and make the most of limited resources.

The six key components, particularly the detailed slope inventory, historical hazard database, and integrated susceptibility / vulnerability / risk mapping and establish a robust foundation for evidence-based maintenance prioritization.

## **2.7 Determination of $c$ and $\phi$ from RocLab**

Rainfall and seismic loading are taken as major catalysts for landslides, particularly common in seismically active and monsoon-impacted areas such as the Himalayas (Hao and Mingtao 2024). Academic research consistently underscores the imperative to comprehend the cumulative and interactive impacts of earthquakes and precipitation (Gowhar et al. 2024). Numerical modeling techniques are essential for integrating these dual factors to forecast landslide occurrences which is subjected to such overburden stresses (Yan et al. 2022; Hao and Mingtao 2024). Finite element methods have been effectively utilized to evaluate slope stability under diverse stress conditions considering the external factors such rainfall infiltration and seismic forces both horizontal and vertical.

## **2.8 Soil Nailing Technique: A Case Study of NM Road Section**

Soil nailing provides a superior stabilization strategy for Nepal's steep cut slopes by minimizing excavation and footprint requirements. Using PLAXIS 2D numerical simulations, researchers assessed the performance of these reinforcements on the NM corridor. Following are their key takeaways from their sensitivity analysis include:

- Soil Nail length is the most significant factor affecting slope stability.
- Rising water tables drastically reduce safety factors, requiring proactive adjustments to design
- The study established clear correlations between water table depth and required nail length, ranging from 7.5 m to 13.5 m for specific groundwater conditions

## **2.9 Stability Assessment of Landslides of Selected Stretches of NM Road**

A study on the NM Road investigated the stability of cut slopes by combining geological mapping with geotechnical analysis (Bhagawan and Krishna 2026). The researchers identified significant variations in rock quality, confirming that the slopes are prone to toppling, wedging, and planar failures. By applying advanced tools like DIPS, SMR, and Q-slope alongside safety factor calculations, the team successfully categorized the stability of different sections (Claudio et al. 2022). Their

findings conclude with site-specific engineering recommendations to effectively mitigate these slope failures.

## 2.10 Climatic Conditions

The NM Highway, located in central Nepal, features a subtropical monsoon climate that strongly influences its ecology and infrastructure. From June to September, the area gets hit with about 80% of its total yearly rain—averaging a massive 2,650 mm. The heavy downpours and the high humidity, the ground stays soaked, which is a recipe for disaster for roads and infrastructure. This constant saturation is exactly what triggers the landslides and instability we see every year.

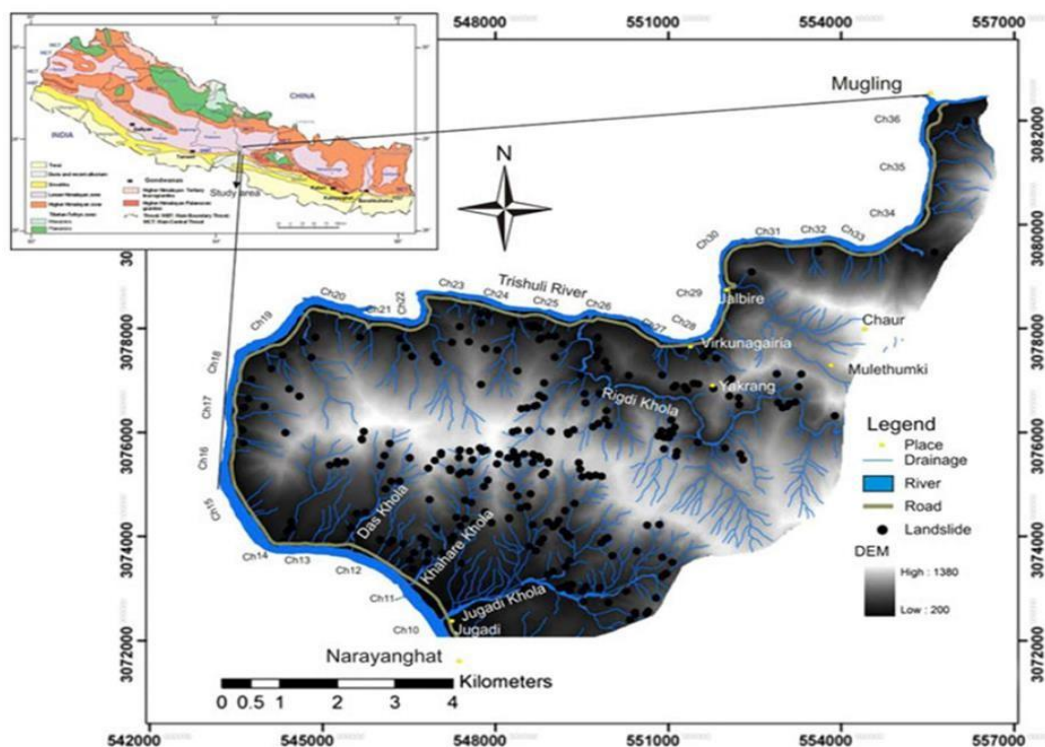


Figure 2-1: Geographical map of the study area (Source: Devkota et al. 2013).

Rainfall varies a lot across the region, but south-facing slopes get hit the hardest. These intense storms trigger fast runoff that washes away soil and moves massive amounts of sediment. Because the terrain is so steep, this erosion quickly turns into landslides that threaten the highway and make maintenance a constant battle (Eltaher et al. 2026).

During the monsoon, the road is under constant threat, often stranding travelers and cutting off transportation. Between the heavy rain and the rugged landscape, it's

clear we need much tougher infrastructure and better slope management to keep the highway safe and open year-round.

## 2.11 Slope Stability Analysis

In this section the plate tectonics concept, seismicity, and numerical methods used in slope stabilization measures are as follows:

### 2.11.1 Plate tectonic concept

From the geological studies, the force that moves our continents comes from the collision between the Indian and Eurasian plates. During this move, the Indian plate sinks below the Eurasian plate, which in turn rises higher and higher. As a result, Peter and Topponnier considered Tibet as a pressure gauge of Asia. Indian plate applies pressure to Eurasia and Tibetan plateau rose to present position and is still rising at the rate of 3.2 cm to 12.7 cm a year. In general, the Tectonic Forces have created linear belt of cracks parallel to Himalayas along with the rise of height of mountains.

Due to Tectonic movement, Himalayan area and its neighboring areas are seismically very active. Most of epicenters of earthquakes are found to be located in the unstable zones. The Nepal lie in the Himalaya along with major tectonic boundary and various longitudinal zones of the Himalaya are shown in Figure 2.2. Seismic hazard map of Nepal shows that earthquake with a peak acceleration of 0.3 - 0.5 g may occur in 50 years, making the country very vulnerable to earthquake. A recent earthquake of magnitude 7.6 on Richter scale also proves that the country is highly vulnerable to earthquake.

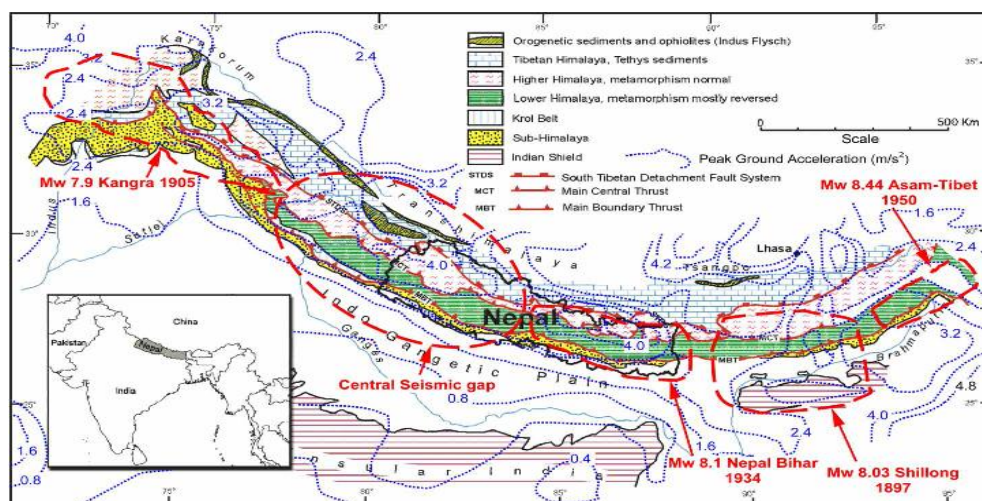


Figure 2-2: Major tectonic boundary and various longitudinal zones.

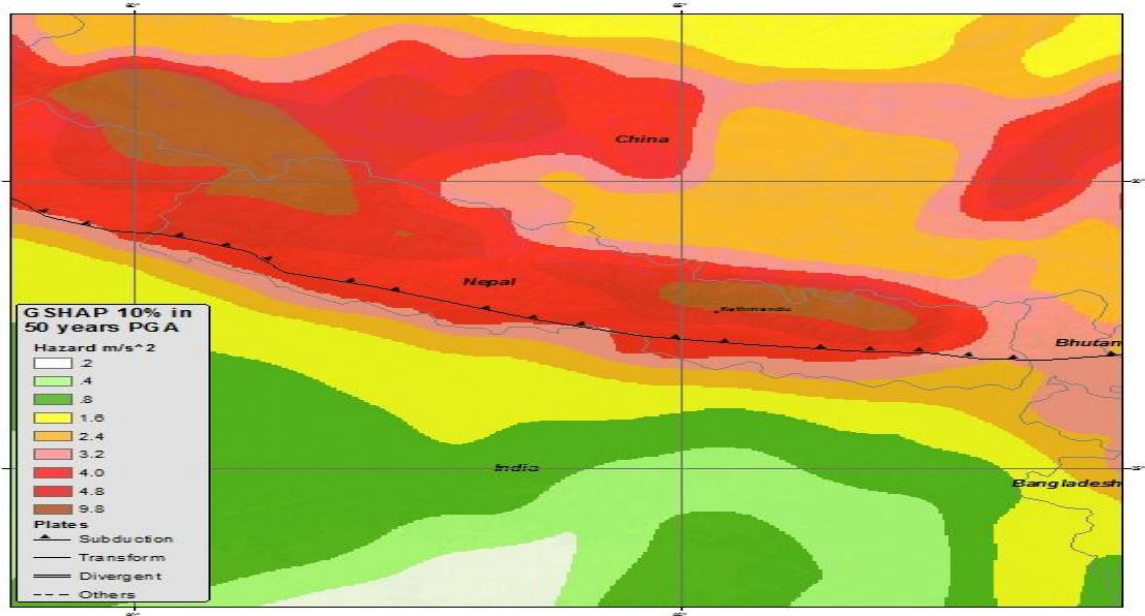


Figure 2-3: Seismic hazard map of Nepal (source: USGS).

### 2.11.2 Seismicity

Nepal Himalaya is considered to be located in seismically active zone. There had many earthquakes with epicenter located within the region. The most renowned earthquakes are 1934 Nepal-Bihar Earthquake and 1987 Nepal Earthquake which has intensity of 8.2 and 7.2 in Richter scale respectively. They had caused uncountable number of damages.

From east to west, the seismic activity is related to the geo-tectonics of Himalayan Geology. The southern Indo-Gangetic tectonic plate is sub ducting into the northern Tibetan (Eurasian) Tectonic plate since the Miocene Period. The seduction rate is presumed to be at the rate 5-8cm/year. The collision of two continental tectonic plates has caused the rise of the Himalaya, which is the highest mountain range in the world. As a result, major longitudinal NW-SE striking thrust faults are also created. These are Main Frontal Thrust (MFT) between Indogangetic Plain and the Siwaliks, the Main Boundary Thrust (MBT) in between the Siwaliks and Higher Himalaya and the Main Central Thrust (MCT) in between the Higher Himalaya and the Higher Himalaya. Nepal is prone to an earthquake of minor or major magnitude (Zhice et al. 2024). The figure 2-4 presents *historical events of earthquakes in Nepal* (Source: Department of mines and Geology (DMG)).

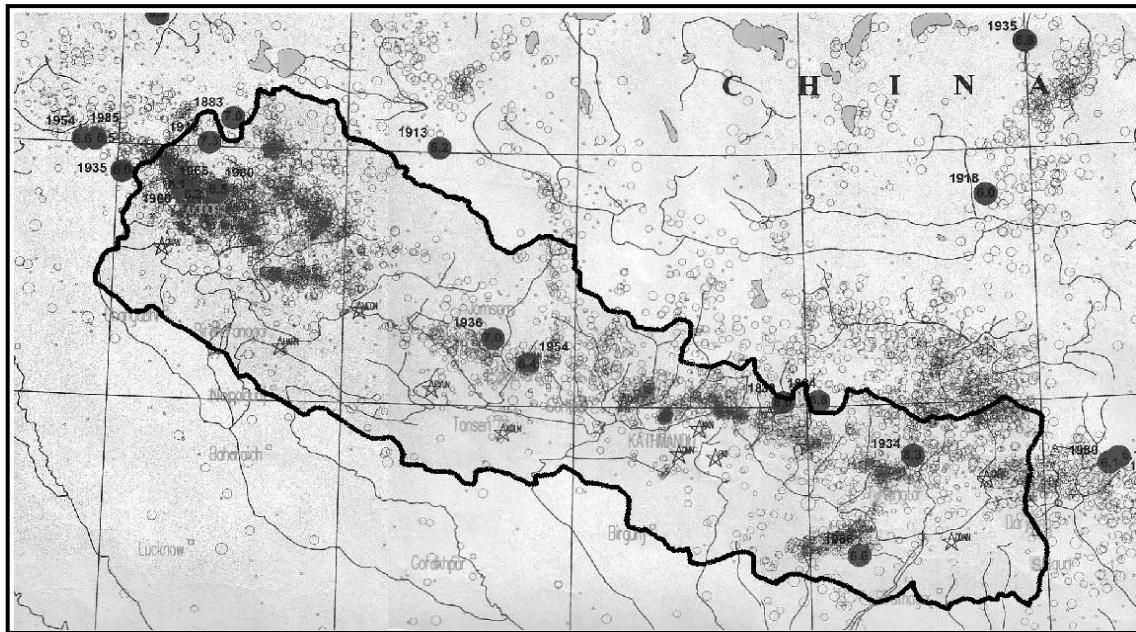


Figure 2-4: Historical events of earthquakes in Nepal.

## 2.12 Numerical Analysis

### Slope Stability Analysis using GeoStudio (SLOPE/W, SEEP/W and QUAKE/W)

The LEM is a traditional and widely used analytical approach for evaluating the stability of slopes in soil and rock. LEM determines the FS of a potential slip surface by comparing the resisting forces (shear strength) with the driving forces (shear stress) that contribute to slope failure. The method assumes that the slope mass is divided into slices and equilibrium equations are applied to estimate the shear resistance along the slip surface. LEM is commonly used in geotechnical engineering for landslide assessment and the design of slope stabilization measures (Yadav et al. 2022).

SLOPE/W is a software tool specialized for performing stability analysis of soil and rock slopes using the LEM. It is extensively applied in geotechnical projects to assess the performance of natural and man-made slopes under static and dynamic loading conditions. SLOPE/W supports several analytical methods, including Bishop, Janbu, Spencer, and Morgenstern–Price, for computing the FS.

#### 2.12.1 General procedure for slope stability analysis in slope/w

Table 2-1: General procedure for slope stability analysis in Slope/w

Step	Description
------	-------------

Model Setup	(a) Geometry Definition: Define slope profile, soil layers, and structural components (e.g., retaining walls, anchors, berms).  (b) Material Properties: Assign rock parameters such as Unconfined Compressive Strength (UCS), Geological Strength Index (GSI), Unit Weight ( $\gamma$ ), etc
Slip Surface & Analysis Settings	Select or automatically generate slip surface shapes (circular/non-circular). Choose preferred LEM algorithm — Bishop, Janbu, Spencer, etc.
Pore Water Pressure & Loading	Assign pore water pressure, phreatic surface, or import groundwater conditions from SEEP/W. Apply loads such as surcharge, seismic forces, or structural loading to represent field conditions.
FS Calculation	SLOPE/W iteratively computes the FS by balancing driving and resisting forces along the slip surface. Sensitivity and probabilistic analyses can be performed for parameter uncertainty evaluation.
Results Interpretation & Post-Processing	Visualize outcomes such as critical slip surface, FS values, stability comparison charts, displacement patterns, and reinforcement forces to determine stabilization requirements.
Deformation and Settlement output	Visualize outcomes such as critical slip deformation and settlement analysis.

### 2.13 Rock Mass Classification

With a varying strength of intact rock and different parameters to explain the discontinuities of rock masses, there is need to have classification for rocks. Some of the rock mass classification systems are as follows: -

- A) Rock Mass Rating (RMR)
- B) Q- System
- C) Geological Strength Index (GSI)

**A) Rock Mass Rating (RMR)**

RMR also known as Geotechnics Classification System, was originally purposed by Bieniawski in 1973 in South Africa. It was slightly modified in 1989. It is a rating done with maximum value 100 based on the parameters given below: -

- a) Strength of intact rock– maximum score of 15
- b) RQD – maximum score of 20
- c) Mean spacing of the discontinuities – maximum score of 20
- d) Condition of discontinuities – maximum score of 30
- e) Groundwater conditions – maximum score of 15
- f) Orientation of discontinuities

The rating for first five parameters are summed up to find RMR value. The RMR value lies in the range 0-100. The last one is the adjustment for RMR value which are negative values from 0 to -60. The rock mass class based on RMR value is shown below.

*Table 2-2: Rock mass class based on RMR*

RMR	81-100	61-80	41-60	21-40	0-20
Class Number	I	II	III	IV	V
Description	Very Good Rock	Good Rock	Fair Rock	Poor Rock	Very Poor Rock

**B) Q-System**

Barton et al. (1974) of Norwegian Geotechnical Institute purposed Q-system of rock mass classification. The objective of Q- System was to characterize the rock mass and determine the support requirements. Similar to RMR, Q-system is also based on six parameters which are listed below:

*Table 2-3: Q- system parameters*

S. No.	Parameter	Symbol	Range
a)	Rock Quantity Designation	RQD	0-100

b)	Joint Set Number	$J_n$	1-20
b)	Joint Roughness Number	$J_r$	1-4
b)	Joint Alternation Number	$J_a$	1-20
b)	Joint Water Reduction Factor	$J_w$	0.1-1
b)	Stress Reduction Factor	SRF	1-20

Q- Value is given as: -

$$Q = \left(\frac{RQD}{J_n}\right) \left(\frac{J_r}{J_a}\right) \left(\frac{J_w}{SRF}\right) \dots \dots \dots \text{Equation 2.13.1}$$

The Q- Value lies on logarithmic scale from 0.001 to 1000 indicating exceptionally poor to exceptionally good rock.

*Table 2-4 Rock mass classification based on Q- system*

Q- Value	Class	Rock Mass Quality
400-1000	A	Exceptionally Good
100-400	A	Exceptionally Good
40-100	A	Very Good
10 - 40	B	Good
4 - 10	C	Fair
1 - 4	D	Poor
0.1 - 1	E	Very Poor
0.01 - 0.1	F	Extremely Poor
0.001- 0.01	G	Exceptionally Poor

### C) Geological Strength Index

The Hoek – Brown Failure Criteria is popular in studying stability of rock mass in underground structure. The failure criterion is expressed in Equation 13-1 Geological Strength Index.

$$\sigma'_{1f} = \sigma'_{3f} + \sigma_{ci} \left( m_m \frac{\sigma'_{3f}}{\sigma_{ci}} + s \right)^a \dots \dots \dots \text{Equation 2.13.2}$$

Where,

$\sigma'_{1f}$  = effective major principal stress at failure

$\sigma'_{3f}$  = effective minor principal stress at failure

$\sigma_{ci}$  = uniaxial compressive strength at intact rock.

The constant a and s depend on rock mass characteristics.

GSI is estimated from RMR value as shown in equation 2.13.3.

$$GSI = RMR_{76} = RMR_{89} - 5 \dots\dots\dots \text{Equation 2.13.3}$$

RMR<sub>89</sub> is according to Bieniawski (1989) and RMR<sub>76</sub> is computed as per Bieniawski (1976) with maximum rating for ground water condition is 10.

GSI is a recent rock mass classification developed by Hoek (1994) based on geological observation rather than the numerical value.

GSI value for jointed rock mass is as follows: -


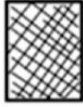




GEOLOGICAL STRENGTH INDEX FOR JOINTED ROCKS		SURFACE CONDITIONS					
		VERY GOOD	GOOD	FAIR	POOR	VERY POOR	
STRUCTURE		DECREASING SURFACE QUALITY →					
	INTACT OR MASSIVE—intact rock specimens or massive in situ rock with few widely spaced discontinuities	DECREASING INTERLOCKING OF ROCK PIECES ↓	90				
	BLOCKY—well interlocked undisturbed rock mass consisting of cubical blocks formed by three intersecting discontinuity sets		80	70			
	VERY BLOCKY—interlocked, partially disturbed mass with multi-faceted angular blocks formed by 4 or more joint sets		60				
	BLOCKY/DISTURBED/SEAMY—folded with angular blocks formed by many intersecting discontinuity sets. Persistence of bedding planes or schistosity		40				
	DISINTEGRATED—poorly interlocked, heavily broken rock mass with mixture of angular and rounded rock pieces		20				
	LAMINATED/SHEARED—Lack of blockiness due to close spacing of weak schistosity or shear planes		10				

Figure 2-5: GSI for jointed rocks.

The Hoek- Brown parameter for rock mass and intact rock is given as Hoek- Brown parameter for rock mass in Equation 13-2.

$$m_m = m_i \exp\left(\frac{GSI-100}{28}\right) \text{ For } GSI > 25 \dots\dots\dots\text{Equation 2.13.3}$$

Where  $m_i$  is specific to rock type.

For good quality rock mass ( $GSI > 25$ )

$$a = 0.5$$

$$s = \exp\left(\frac{GSI-100}{9}\right)$$

For poor quality rock mass ( $GSI < 25$ )

$$a = 0.65 - \frac{GSI}{200}$$

$$s = 0$$

### 3. STUDY AREA AND DATA

#### 3.1 General Geology, Geomorphology

This area traverses a critical geological boundary

- Main Boundary Thrust (MBT): This major active thrust fault separates two distinct geological zones: the younger, less metamorphosed Siwalik Group (Sub-Himalayan Zone) to the south from the older, metamorphosed Lesser Himalayan Sequence (Mahabharat Range) to the north. The movement along this thrust system is the primary driver of the region's instability and frequent seismic activity.
- Other Thrusts and Faults: The area is fragmented by numerous local faults and folds, which further weaken the rock mass.



*Figure 3-1: Tuin khola area (CH 33+900), before massive landslide.*

As per geological map of Nepal, the NM Road (NH44) is entirely located within the Chitwan district of Nepal. The area at the Tuin tunnels, specifically near Tuin khola is part of Ichchhakamana Rural Municipality-6, where thinly foliated dark gray phyllite and thin to thick foliated quartzite are encountered. Geological map of

Nepal with the project area is shown in Figure 3-2. This Geological map of Nepal was published by Dahal 2008.

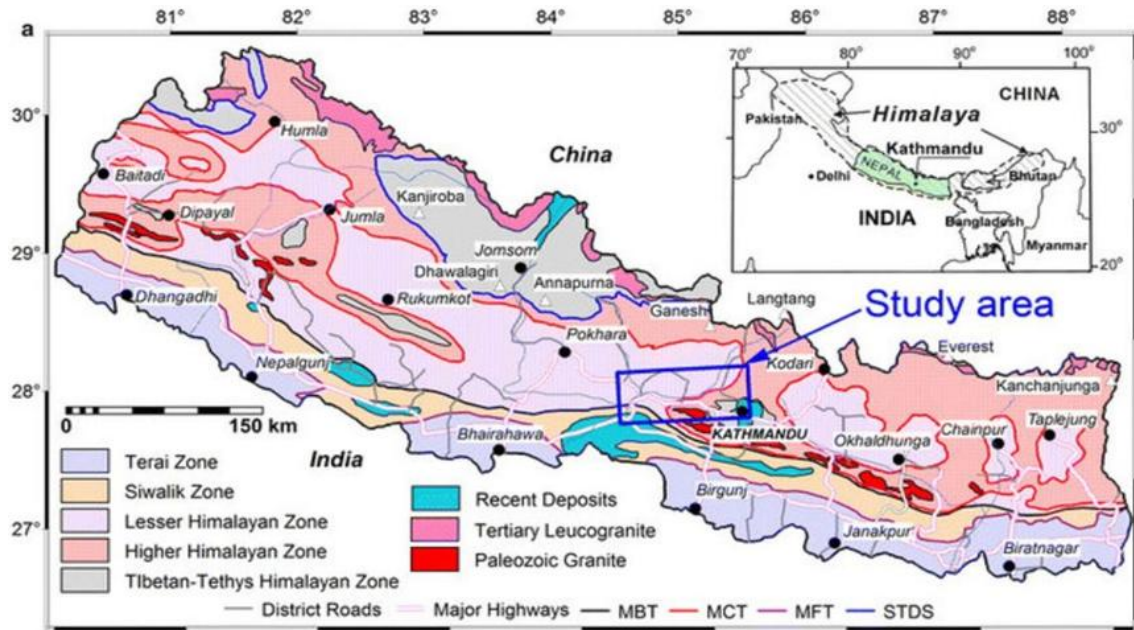


Figure 3-2: Geological map of Nepal with the study area.

The road section traverses both the Siwaliks in the south and the Lesser Himalaya in the north. The rocks encountered in this specific area belong to the Lesser Himalayan sequence and primarily consist of thinly foliated dark gray phyllite and thin to thick foliated quartzite.

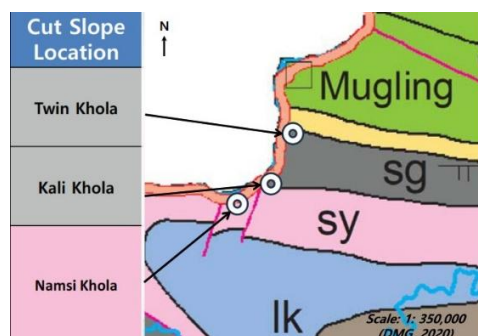


Figure 3-3: Geological setting NM region (Source: DMG, 2020).

The geological formation of Tuin khola area is Sangram Formation (sg) in which thinly foliated, phyllite occasionally interfoliated with thin to thick bands of quartzite and limestone.

The instability is exacerbated by geological structures like faults, joints, and steep, cut slopes, with many landslides being reactivation of older slides. The figure

illustrates the Digital Elevation Model (DEM) of the tuin khola with contour lines indicating terrain variation.

### 3.2 Slope Characteristics

In reference to Figure 3-4, the slope morphology shows a clear contrast between the right and left banks of the Tuin Khola section. The right bank (red-orange zone) is characterized by very steep slopes with dense contour spacing, indicating a high gradient terrain composed of exposed or highly weathered rock. This area is highly susceptible to instability and is identified as a landslide-prone zone, with significant potential for rockfall and debris flow initiation. In contrast, the left bank (green-yellow zone) exhibits relatively gentler slopes with wider contour spacing, suggesting lower gradient conditions and comparatively higher stability. The elevation gradually increases outward on both sides of the channel; however, the left bank terrain appears more stable, possibly underlain by weaker but more uniform lithology such as phyllite or schist, which is commonly found in the hill regions of Nepal.

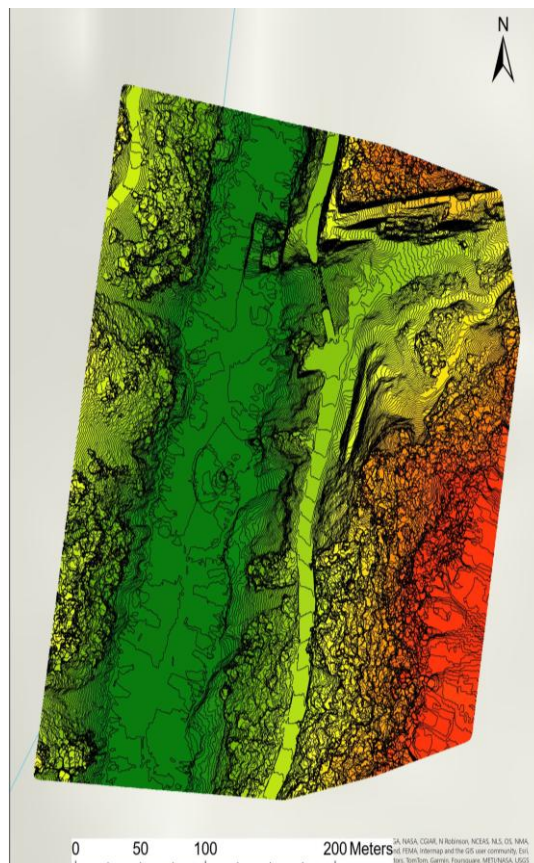


Figure 3-4: Digital elevation model of tuin khola area.

## 4. METHODOLOGY

The research methodology of this study consists of work flow, study steps, methods of gathering information and data analysis approaches.

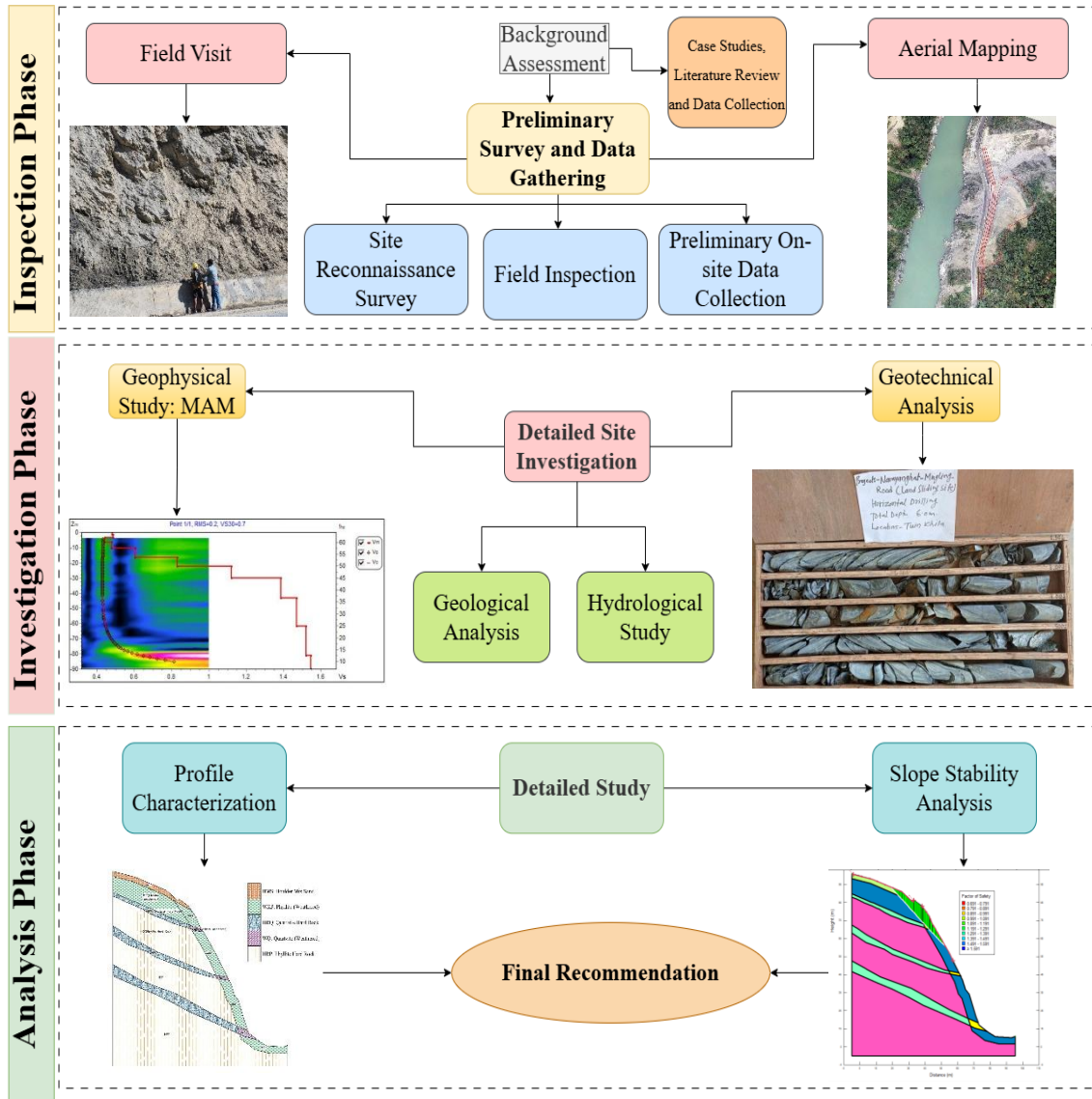


Figure 4-1: Overall methodology of the study.

### 4.1 Inspection Phase

#### 4.1.1 Site reconnaissance survey

The reconnaissance survey kicked off with its geology, water flow, and rock conditions affecting the slopes at Tuin Khola. This area is well known as one of the region's biggest landslide hotspots, with the slopes giving way repeatedly especially when the monsoon rains hit.

The observations are:

- To identify visible signs of slope instability
- To point out features such as bedding planes, joints, faults, and drainage lines
- To assess the influence of construction activities like road cutting, excavation, deforestation, and drainage modifications on slope behavior
- To plan representative locations for detailed geological, geotechnical and geophysical studies

#### 4.1.2 *Field observations*

- The Tuin Khola area lies within the Lesser Himalayan zone, consisting mainly of highly weathered schist, phyllite, and quartzite.
- The slopes which are steep up to 70°
- In slope there are multiple active and reactivated slides along both cut and natural slopes
- The slope faces show signs of tension cracks, tend to fall down the huge rock mass
- Surface runoff and inadequate roadside drainage were contributing to water accumulation near slope toes
- Seepage lines and small springs were noted at more than 10 locations
- The surface is highly weathered, joint sets, foliations, and minor faults exhibited orientations to instability
- Sparse vegetation was observed on the upper slopes due to previous cuttings and deforestation, reducing root reinforcement and increasing erosion susceptibility
- The widening of the highway has created over-steepened cut slope beyond the natural angle of repose

#### 4.1.3 *Preliminary on site data collection*

- The reconnaissance findings confirm that the Tuin Khola slope instability is primarily due to a combination of fragile geology, heavy monsoonal rainfall.
- The various rock joint sets, weak rock masses, no provision of drainage and road extension has led to slope instability.

#### 4.1.4 *Geological mapping*

Detailed geological map of landslide area in detail to pin out the rock types and structural features like faults, folds, and joint patterns. The RMR value of five sections from ch: 000+020 to ch: 000+0120 along the slope helps to identify the potentially

failure zone. This report identified layers of dark gray phyllite and quartzite, which provide fresh idea about field condition.

#### 4.1.5 Location Logging and sampling

Boreholes drilled at selected locations (as indicated by drilling site photos, e.g., horizontal drilling of 6 m depth). Drilling is done to locate any hidden cracks or foliation planes within the rock mass. We collected both disturbed and undisturbed soil and rock samples to a depth of 6 meters. By targeting the top, middle, and bottom of the landslide, get a complete picture of the ground across the entire failure zone. The resulting borehole logs gave us all the data needed on rock types, water levels, and rock quality (RQD).

Table 4-1: Rock mass data collection on site

<b>BH No</b>	<b>sampling</b>	<b>Started</b>	<b>Finished</b>	<b>Easting</b>	<b>Northing</b>
1	Location-1	2082/05/17	2082/05/17	27.842978	84.558697
2	Location-2	2082/05/17	2082/05/17	27.842706	84.558641
3	Location-3	2082/05/17	2082/05/17	27.842442	84.558582
4	Location-4	2082/05/17	2082/05/17	27.842278	84.558557
5	Location-5	2082/05/17	2082/05/17	27.841976	84.558454

#### 4.1.6 Rock mass characterization

A detailed RMR at five points along the cut slopes of the Slope failures near Tuin Khola. This involves checking everything from the rock's strength and quality (RQD) to the specific details of its joints like how far apart they are, how rough or smooth, and what kind of material is filling in between them. We'll also look closely at how much the rock has weathered. This approach is used for quantifying the rock mass quality and understanding its details logging on kinematic slope stability analysis.

#### 4.1.7 Ground water table monitoring

The ground water table point out at various elevation. The ground water table may fluctuate depending upon the variation of season, draw down and recharging of ground water. The groundwater plays very important role for the analysis and design of foundation. I point out the water seeping area correctly and noted. This piezo metric line of ground water table has been analyzed. The groundwater was encountered in a point, observe and record any movement in the level of the groundwater.

#### 4.1.8 Hydrological study

Rainfall serves as the primary determinant for assessing landslide hazard potential. Situated within the Lesser Himalaya, where rainfall is frequent, and the project site experiences significant rainfall variability typical of Nepal. Approximately 10% of the annual precipitation may occur within a single day, with 50% concentrated within a 10-day span during the monsoon season. Given the heightened precipitation levels during the monsoon season, the likelihood of landslide events is considerably elevated.

### 4.2 Investigation Phase

#### 4.2.1 Geophysical study

Geophysical investigations i.e. MAM test is done by *Explorer Geophysical Consultants Pvt. Ltd*, Tokha, Kathmandu. The results obtained are analysis. These results are necessary to assess the subsurface conditions and evaluate the stability of the roadside cut slope. MAM-1 was carried out at cut slope of road whose exact location is shown in Google Earth Image. Figure 3.2.8.1 shows dispersion curve and S-wave velocity map along this profile. Velocity depth profile for MAM has depicted different layers. Top layer with shear wave velocity of 443 m/s - 481 m/s up to the depth of 6.5 m indicates colluvial deposit with gravels and boulders. The equivalent allowable bearing capacity of this layer is 223 kN/m<sup>2</sup> - 245 kN/m<sup>2</sup>.

Shear wave velocity ranging between 479 m/s - 605 m/s up to the depth of 16.46 m indicates highly weathered and fractured bedrock of phyllite with quartzite and limestone. The equivalent allowable bearing capacity of this layer varies from 239 kN/m<sup>2</sup> - 302 kN/m<sup>2</sup>. Shear wave velocity ranging between 605 m/s - 1122 m/s up to the depth 30.03 m indicates fractured to competent bedrock of phyllite with quartzite and limestone. The equivalent allowable bearing capacity of this layer is 302 kN/m<sup>2</sup> - 589 kN/m<sup>2</sup>. Shear wave velocity ranging between 1386 m/s - 1521 m/s below the depth of 30.03 m indicates competent bedrock of phyllite with quartzite and limestone. The equivalent allowable bearing capacity of this layer is 589 kN/m<sup>2</sup> - 897 kN/m<sup>2</sup>.

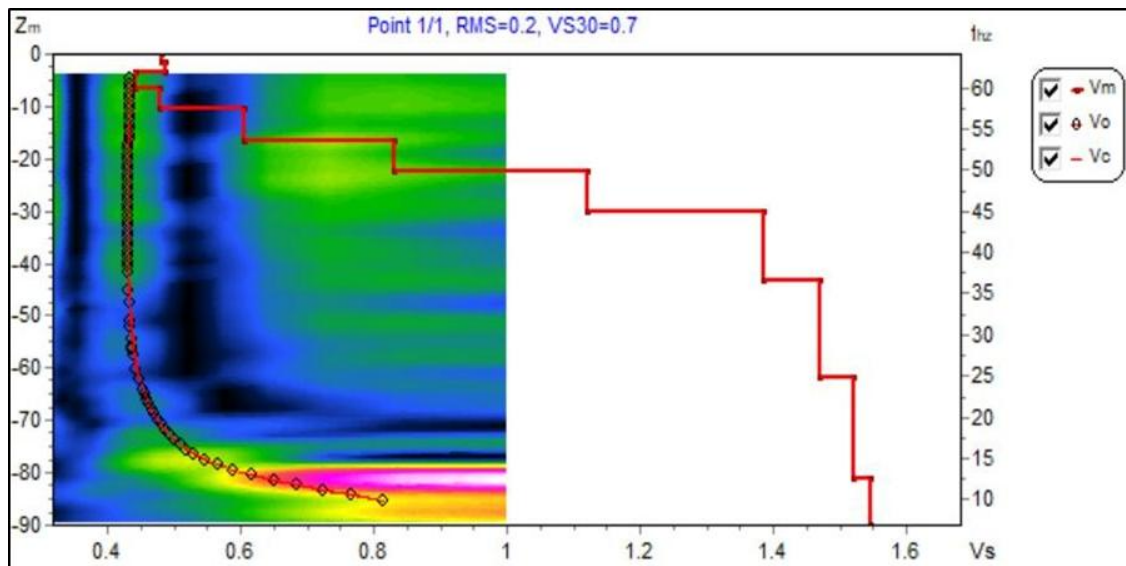


Figure 4-2: Dispersion curve of MAM.

#### 4.2.2 Geological assessment of slopes

Location 1: RMR at the Tuin Khola Cut Slope



Figure 4-3: Exposure of rock mass at L1.

Table 4-2: Works RMR calculation (Bieniawski 1989) at L1

Location /GPS: 27. 842978 ,84.558697					
Attitude of the Foliation (Strike/Dip Amount/Dip Direction): 85°/46°/175°					
Hill slope dip / hill slope dip direction: 80°/280°					
S.No.	Parameter	Joint 1 (J1) – Foliation Plane	Joint 1 (J2)	Joint 1 (J3)	Joint 1 (J4)

1	Attitude (Dip direction/amount)	175°/46°	275°/71°	230°/16°	350°/80°
2	UCS (MPA)	50 - 100			
3	RQD %	Up to 25			
4	Spacing	Lamina-8 cm, metabasic-2 m	8 - 20 cm	6-18 cm	3-11 cm
5	Persistence	>10 m	12-35 cm	20-25 cm	17-38 cm
6	Separation	3-5 mm	2-3 mm	2 mm	1 mm
7	Roughness	Slightly rough	Slightly rough	Slightly rough	Slightly rough
8	Infill Material	clay	clay	clay	Clay
9	Weathering	Slight to moderately	moderately	moderately	Moderately
10	Ground water	Dry	Dry	Dry	Dry

Table 4-3: RMR calculation of failure slope L1

UCS	RQD	Spacing	Conditions of Discontinuities					Ground Water	Basic RMR	SMR
			Persistence	Separation	Roughness	Infilling Material	Weathering			
7	4	7	5	1	3	2	3	15	47	34

Table 4-4: Joint sets and mean joint from random joints at L1

S. N.	Dip Amount	Dip Direction	Remarks
1	46°	175°	Foliation Plane (J1)
2	71°	275°	Joint 2 (J2)
3	16°	230°	Joint 3 (J3)
4	80°	350°	Joint 4 (J4)

5	20°	250°	
6	45°	280°	
7	46°	75°	
8	47°	321°	
9	15°	200°	
10	26°	250°	

Location 2: RMR at the Tuin Khola Cut Slope



Figure 4-4: Exposure of rock mass at L2.

Table 4-5: Works RMR calculation (Bieniawski 1989) at L2

Location /GPS: 27.842706, 84.558641					
Attitude of the Foliation (Strike/Dip Amount/Dip Direction): 90°/70°/180°					
Hill slope dip / hill slope dip direction: 80°/280°					
S.N.	Parameter	Joint 1	Joint 2	Joint 3	Joint 4
1	Attitude (Dip	180°/70°	280°/60°	300°/24°	350°/72°

	Direction/Amount)				
2	UCS (MPA)	10.0-25.0			
3	RQD %	<25			
4	Spacing	1mm-3cm	5cm	8cm	8-10cm
5	Persistence	>25m	5-7cm	2-9cm	1-12cm
6	Separation	1mm	2mm	3mm	1mm
7	Roughness	smooth	Slightly. R	Slightly. R	Slightly. R
8	Infill Material	clay	clay	clay	clay
9	Weathering	Moderately	Moderately	Moderately	Moderately
10	Ground water	dripping	dripping	dripping	dripping

Table 4-6: RMR calculation of failure slope L2

UCS	RQD	Spacing	Conditions of Discontinuities					Ground Water	Basic RMR	SMR
			Persistence	Separation	Roughness	Infilling Material	Weathering			
2	3	7	4	1	3	2	3	4	29	14

Table 4-7: Joint sets and mean joint from random joints at L2

S. N.	Dip Amount	Dip Direction	Remarks
1	70°	180°	J1 (Foliation Plane)
2	60°	280°	J2
3	24°	300°	J3
4	72°	350°	J4

Location 3: RMR at the Tuin khola cut slope



Figure 4-5: Exposure of rock mass at L3.

Table 4-8: Works RMR calculation (Bieniawski 1989) at L3

Location /GPS: 27.842442, 84.558582				
Attitude of the Foliation (Strike/Dip Amount/Dip Direction): 90°/70°/80°				
Hill slope dip / hill slope dip direction: 82°/280°				
S.No.	Parameter	Joint 1 (J1) Foliation Plane	Joint 2 (J2)	Joint 3 (J3)
1	Attitude (Dip Direction/Amount)	180°/70°	305°/73°	40°/69°
2	UCS (MPA)	10.0-25.0		
3	RQD %	25		
4	Spacing	Lamina-5cm	5-7cm	9-17cm
5	Persistence	>25	3-5cm	6-9cm
6	Separation	>7mm-1mm	1-2m	1mm

7	Roughness	S.R -smooth	S. R	S. R
8	Infill Material	clay	clay	clay
9	Weathering	Moderately	Moderately	Moderately

Table 4-9: RMR calculation of failure slope at L3

UCS	RQD	Spacing	Conditions of Discontinuities					Ground Water	Basic RMR	SMR
			Persistence	Separation	Roughness	Infilling Material	Weathering			
2	3	7	4	1	3	2	3	15	40	24

Table 4-10: Joint sets and mean joint from random joints at L3

S. N.	Dip Amount	Dip Direction	Remarks
1	70°	180°	J1 (Foliation Plane)
2	73°	305°	J2
3	76°	300°	J2
4	69°	040°	J3
5	15°	290°	
6	64°	135°	
7	40°	330°	
8	80°	295°	
9	54°	260°	
10	68°	305°	
11	20°	300°	
12	45°	275°	

Location 4: RMR at the Tuin khola cut slope



Figure 4-6: Exposure of rock mass at L4.

Table 4-11: Works RMR calculation (Bieniawski 1989) at L4

Location /GPS: 27.842278, 84.558557					
Attitude of the Foliation (Strike/Dip Amount/Dip Direction): 90°/70°/180°					
Hill slope dip / hill slope dip direction: 82°/280°					
S.N	Parameter	(J1 - Foliation Plane)	Joint 2	Joint 3	Joint 4
1	Attitude (Dip direction/Amount)	180°/70°	265°/70°	285°/21°	345°/76°
2	UCS (MPA)	10.0-25.0			
3	RQD %	25			

4	Spacing	Lamina- 10cm	8-21cm	7-27cm	3-35cm
5	Persistence	<20m	25cm	11cm	21cm
6	Separation	1-5mm	1-2mm	1-2mm	1-2mm
7	Roughness	S.R smooth	S. R	S. R	S. R
8	Infill Material	clay	clay	clay	clay
9	Weathering	Moderatel y	Moderatel y	Moderatel y	Moderatel y
10	Ground water	dry	dry	dry	dry

Table 4-12: RMR calculation of failure slope at L4

UCS	RQD	Spacing	Conditions of Discontinuities					Ground Water	Basic RMR	SMR
			Persistence	Separation	Roughness	Infilling Material	Weathering			
2	3	7	4	1	3	2	3	15	40	26

Table 4-13: Joint sets and mean joint from random joint at L4

S. N.	Dip Amount	Dip Direction	Remarks
1	70°	180°	J1 (Foliation Plane)
2	70°	265°	J2
3	21°	285°	J3
4	76°	345°	J4

Location: 5: RMR at the Tuin khola cut slope



Figure 4-7: Exposure of rock mass at L5.

Table 4-14: Works RMR calculation (Bieniawski 1989) at L5

Location /GPS: 27.841976, 84.558454					
Attitude of the Foliation (Strike/Dip Amount/Dip Direction): 90°/70°/180°					
Hill slope dip / hill slope dip direction: 80°/280°					
S.N.	Parameter	(J1 -Foliation Plane)	Joint 2	Joint 3	Joint 4
1	Attitude (Dip direction/Amount)	180°/70°	269°/73°	285°/25°	355°/76°
2	UCS (MPA)	10.0-25.0			
3	RQD %	25			
4	Spacing	Lamina-15cm	5-9cm	6-13cm	13-21cm
5	Persistence	<20m	19cm	11cm	8cm
6	Separation	>5mm	2-3mm	2-5mm	3-5mm

7	Roughness	S.R -smooth	S. R	S. R	S. R
8	Infill Material	clay	clay	clay	clay
9	Weathering	Moderately	Moderately	Moderately	Moderately
10	Ground water	Lower damp	Lower damp	Lower damp	Lower damp

Table 4-15: RMR calculation of failure slope at L5

UCS	RQD	Spacing	Conditions of Discontinuities					Ground Water	Basic RMR	SMR
			Persistence	Separation	Roughness	Infilling Material	Weathering			
2	3	7	4	1	3	2	3	10	35	22

Table 4-16: Joint sets and mean joints from random joints at L5

S. N.	Dip Amount	Dip Direction	Remarks
1	70°	180°	J1 (Foliation Plane)
2	73°	269°	J2
3	25°	285°	J3
4	76°	355°	J4

#### 4.2.3 Hydrological assessment

The infiltrate water from the crown portion on to the slope area was clearly observed and it was main reason for increasing the landslide triggering force. The groundwater levels can vary and are subjected to seasonal fluctuations in response to major weather events.

Due to the presence of fine sediment pockets at different levels at bottom part, these layers were saturated for the long time and is triggering the landslide mostly.

The slope stability analysis was conducted by considering the effect of rainfall, since the groundwater table was not encountered during drilling and MAM testing. An extreme rainfall intensity typical for monsoon conditions in the study area was used in the analysis. For Chitwan District and similar inner-Terai regions of Nepal, peak rainfall intensities during extreme monsoon events can be substantially higher than

nominal values, with local warning criteria considering up to approximately 60 mm in 1 hour as hazardous for landslides and flooding. Therefore, a conservative extreme hourly intensity of 60 mm/hour was adopted in the stability analysis to simulate intense monsoon rainfall likely to trigger slope failure.

#### 4.2.4 Geotechnical study

Geotechnical test is done by *Peso Consultants Pvt. Ltd., Shankhamul, Kathmandu*. The results obtained by *Peso Consultants* are used for numerical modelling in present study. These results are necessary to assess the subsurface conditions and evaluate the stability of the roadside cut slope. Geotechnical horizontal drilling is a specialized technique used for investigating, stabilizing, and monitoring rock slopes, horizontal drilling is carried out laterally into the rock mass, typically perpendicular to the slope face, allowing us to evaluate and improve the internal mechanical condition of potentially unstable rock masses.

It assess internal rock mass conditions such as joints, fractures, weathered zones, and lithology, as well as groundwater behavior. It enables installation of drainage systems to lower pore water pressure, placement of reinforcement elements like rock bolts, s, or soil nails, and performance of in-situ tests including Lugeon tests, geophysical logging, and monitoring instrumentation. The drilling direction is usually oriented normal to the expected failure plane and planned considering slope geometry, mode of failure, hole diameter and length, and site accessibility.



Figure 4-8: Core samples recovered at Tuin khola.

*a) Debris soil sample*

For BMS containing sand, gravel, and boulders, the laboratory tests depend on particle size and soil behavior. Because this type of material is highly heterogeneous, a combination of coarse-grained and fine-grained soil tests is required. Following are the tests has been done in CMTL lab:

- Particle size distribution
- Specific Gravity
- Natural moisture content
- Direct shear test
- Bulk density/unit weight
- Modified / Standard Proctor Test



*Figure 4-9: Debris samples for sieve analysis.*

*b) Rock test from debris block and core sample*

Rock properties play a vital role in determining the suitability of rock material for civil engineering works such as foundations, tunnels, bridges, dams, pavements and slope stability. Laboratory tests are conducted to evaluate physical, mechanical and durability characteristics of rock, UCS tests done to quantify their unconfined

strength Geological Report RMR of the Cut Slopes along the NM Road NH44 Highway at the Tuin khola.

Following are the tests has been done in CMTL lab:

- Uniaxial Unconfined Compressive Strength
- Point Load Index Test
- Water Absortion and Specific Gravity
- Direct Shear Test
- Bulk density/dry density



*Figure 4-10: UCS test of rock sample.*

### **4.3 Analysis Phase**

#### *4.3.1 Detailed study*

The landslide triggering by road construction in Nepal involves:

- Excessive cutting heights and steep slopes

- Absence of slope protection measures
- Lack of drainage systems
- Side-casting of excavated spoil, and
- Vegetation removal.
- The Department of Roads guidelines recommend slope cuts of 1:0.3 for hard rock and 1:0.8 for soils

#### 4.3.2 *Profile characterization*

The geotechnical investigation and profile characterization indicate that the project site consists of layered soil and rock formations comprising topsoil, BMS, weathered rock, and fresh bedrock. The subsurface materials varies with depth.

Weathered rock strata provide moderate bearing support, while fresh bedrock offers competent foundation support for structural loading.

Groundwater seepage and surface runoff are important controlling factors affecting slope stability and excavation safety.

#### 4.3.3 *Slope stability assessment*

For the slope stability analysis, the natural terrain profile after the failure was considered, along with the slope terrain after the proposed mitigation measures, for the analysis and design process. The subsoil layers were defined based on the findings from both geophysical and geotechnical investigations. The water table condition during the monsoon period was estimated based on the present water table recorded from field observations, springs, and seepage levels, as well as the saturation layer observed from geophysical tests.

The geotechnical analysis for the slope stability was done using the GeoStudio 2023 software (SEEP/W, SIGMA/W & SLOPE/W). The most critical sections were selected after numerous work at the field and office. The field work consists of Topographical Survey work, Geophysical testing work, Geotechnical Investigation work etc. Similarly, office work consists of finalization of level of existing landslide crown, road level and the toe area of landslide. SEEP/W was run to analyse the seepage conditions of the slope and SIGMA/W was run to know the stress and displacement of the slope mass under the pore water pressure increasing the landslide triggering force. Finally, SLOPE/W was run to calculate the FS with a proposed mitigation measure to stabilize the landslide. SIGMA/W works on the principles of finite element method and SLOPE/W runs on the principle of LEM.

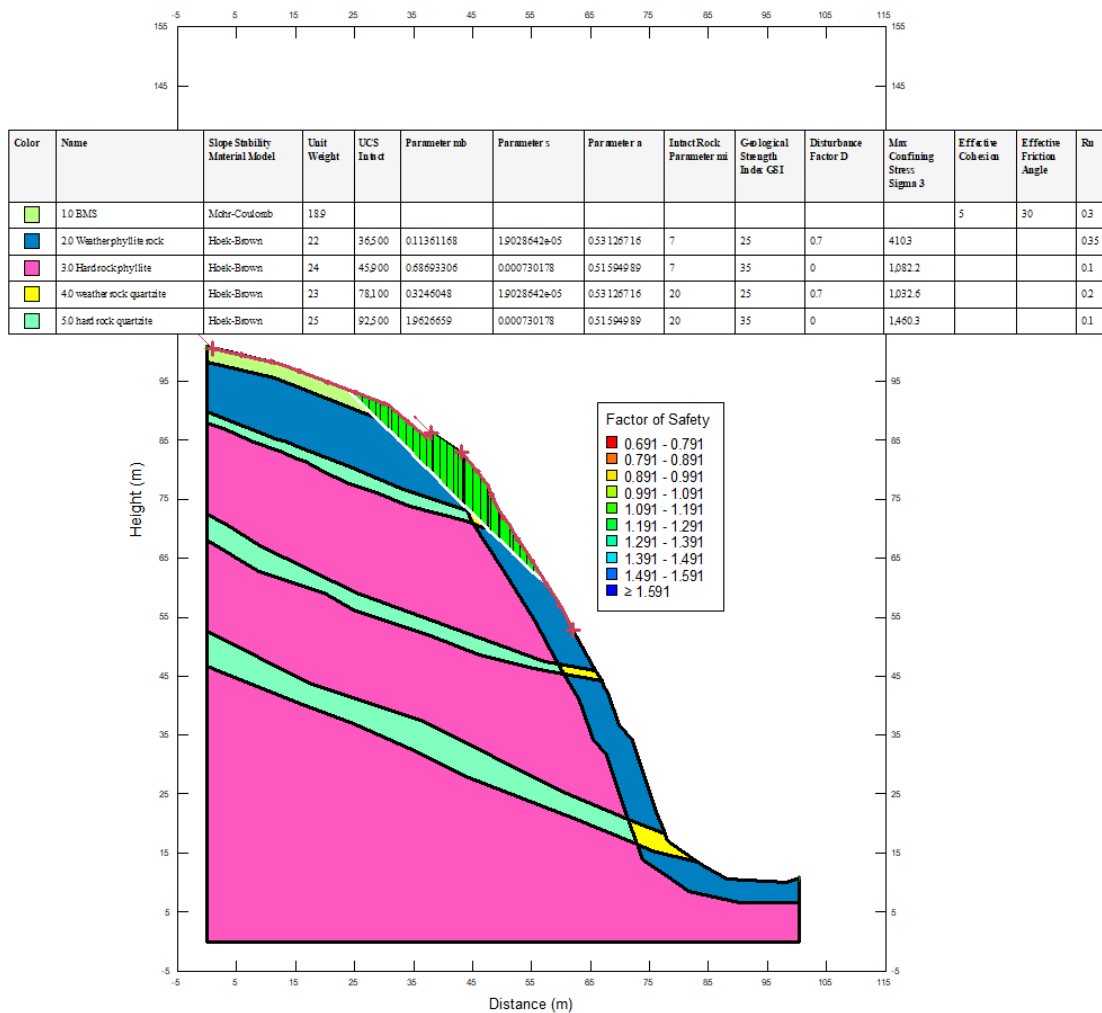


Figure 4-11: Slope stability model in Geo-studio at locations (L3).

- a) *Static Stability Analysis:* The FS has been calculated under initial static conditions, considering the existing slope geometry, the determined material properties, and saturated material phenomena. This analysis establishes the baseline stability of the slope under its self-weight and current environmental conditions.
- b) *Rainfall-Induced Stability Analysis:* The influence of rainfall infiltration on pore water pressures and, consequently, the effective shear strength of the soil is simulated.

The numerical models will give us a clear look at safety factor maps, stress points, deformation patterns, and exactly where the slope is likely to slide. We'll use these results to pinpoint why the landslide is happening and predict how it might move in the future.

#### 4.3.4 *Final recommendation*

Our investigated methods were solid and gave us a clear, reliable picture of the site. By combining field tests, drilling, and lab work, we've gathered all the data we need on soil and rock layers, how the ground will actually behave during monsoon. Specifically, we successfully mapped out the surface debris, weathered layers, and the solid bedrock underneath, all of which are vital for designing slope analysis.

For design and analysis purposes, following are included:

- All structural foundations be designed considering the verified subsurface profile and groundwater condition
- Excavation activities follow staged construction methodology with proper slope protection measures
- Drainage management be incorporated to minimize pore water pressure buildup during monsoon conditions
- Additional detailed investigation and instrumentation monitoring be conducted for critical structures, deep excavations, or high-risk slope sections
- Geotechnical supervision be maintained during construction to verify actual field conditions against interpreted subsurface profiles

## 5. RESULTS AND DISCUSSION

### 5.1 Main Causes for Occurrence of Slide

The causes for occurring the observed several slides are light rainfall for long period, presence of the loose BMS on steep slope at crown part and poor surface and subsurface drainage. Highly weathered and fractured bedrock of phyllite and thin band of quartzite composite the landslide. The slope failure appears to be primarily caused by soil erosion. Additional contributing factors include:

- Activation of a local fault observed on the left side of the slide area, increasing the likelihood of plane failure along the foliation plane
- A steep slope with loose BMS at the crown of the slide
- The natural hill slope and the foliation planes dipping in the same direction, creating an unfavorable geological condition
- Presence of loose materials on the steep slope and exposure of bedrock, indicating an unstable condition
- Signs of active slope movement are visible at the crown area, where surface features suggest continued deformation
- Loss of Toe Support: The most significant cause of the slope instability is the removal of toe support due to back-cutting carried out to create space for the road extension on the slope side of the road
- Disturbance of the Natural Slope Equilibrium
- The site conditions show high susceptibility to wedge failure due to fault activation and BMS at the upper portion of the slope

### 5.2 Geotechnical and Geophysical Analysis

The geotechnical involves field core drilling, core box sampling, borehole logging, and laboratory testing of relevant to determine rock index properties and shear strength parameters. These results were confirmed by geophysical methods such as MAM in which develop continuous subsurface shear-wave velocity profiles and to identify its velocity variations in soft and hard rock layers, groundwater-influenced zones, potential weak strata, etc. This findings enabled to work with more accurate models and its guide to select and design of mitigation measures.

The bore log reveals a subsurface dominated by highly fractured phyllite, a fragile geological structure that significantly elevates the risk of slope failure. Dynamic cone penetration test (DCPT) not performed so their results has not shown in bore log. It's differentiate that the ground is initially weak and "very loose" at the surface gradually becoming "medium dense" as it reaches a depth of 6 meters. while the

ground was recorded as dry during the investigation, the fractured nature of the phyllite makes it highly susceptible to rapid saturation and shear strength loss during heavy monsoon rains, which can trigger sudden collapses or debris flows. Consequently, the core recovery of drilling limited to 25 %. This graph has tentative drawn as per site rock strata which remains a critical vulnerability point for the highway, requiring proactive stabilization measures like subsurface drainage or reprofiling to prevent recurrent road blockages.

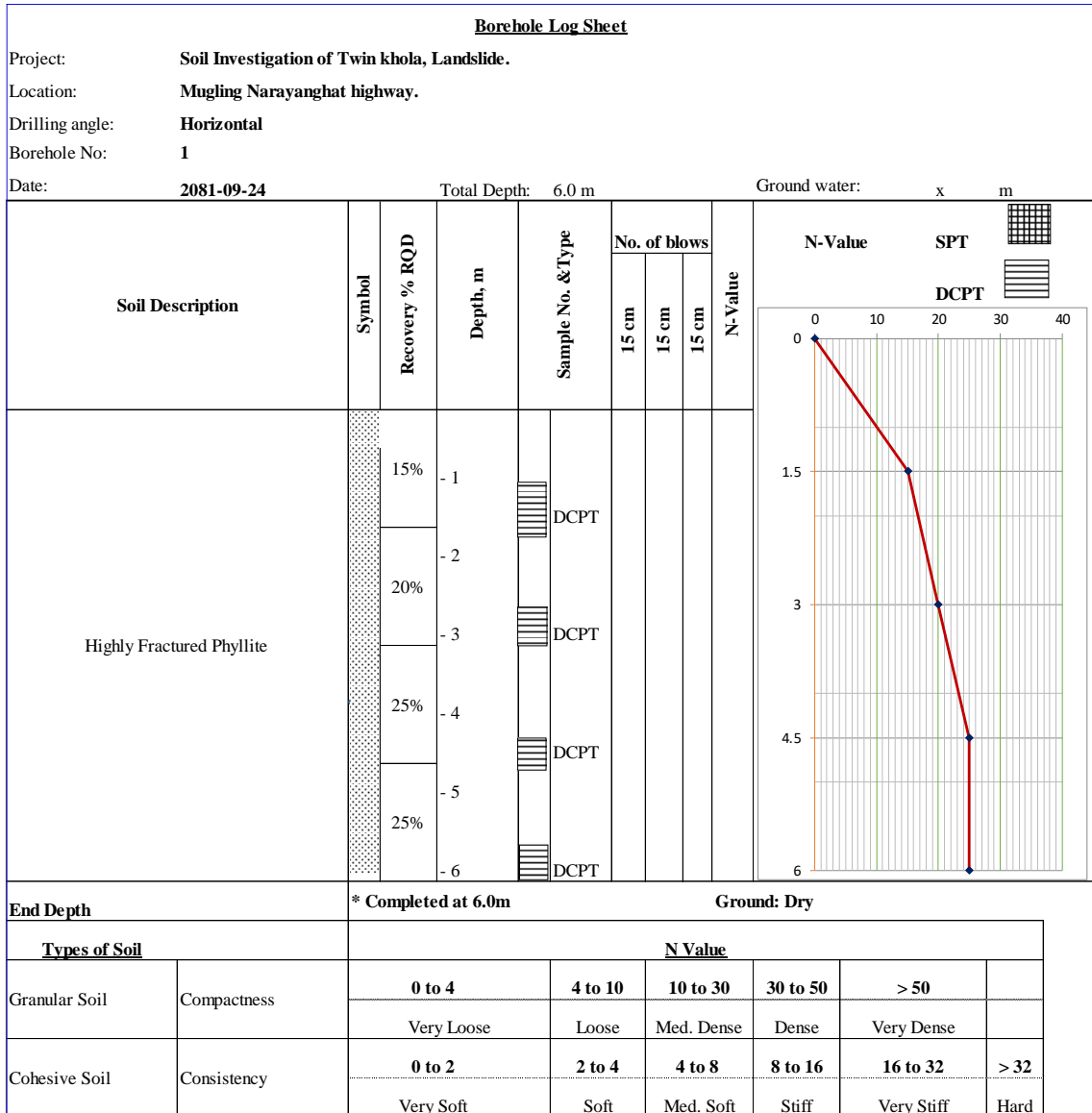


Figure 5-1: Borehole log sheet of horizontal drilling.

Table 5-1: Lab results: colluvial debris & rock block

S.No.	ISCS Classification	Sample No.	Percentage of			NMC	Specific Gravity	Direct shear test		Unit Weight		
			Gravel	Sand	Fines (Silt & Clay)			C	φ			
			%	%	%			%	(G)			
1	GP-SM	1	62.6	26.3	11.0	14.0	2.7	5.3	25.3	17.4		
	GP-SC	2	27.1	65.0	7.9	20.5	2.6	4.7	29.9	18.9		
	GP-SC	3	50.0	39.5	10.4	8.7	2.7	-	-	18.7		
	GP-SC	4	59.4	34.8	5.9	19.0	2.7	0.6	29.2	19.1		
2	GP	1	97.4	2.2	0.5	1.8	2.4	44.3	37.2	27.8		
	GP	2	95.2	4.5	0.2	1.4	2.4	51.4	38.8	26.6		

Table 5-2: Laboratory test data of core and rock block sample

Rock sample	ISCS Classification	Pit of Test sample, m	Percentage of			NMC	Specific Gravity	Direct shear test		UCS	PI	Unit Weight
			Gravel	Sand	Fines (Silt & Clay)			C	φ			
			%	%	%			%	(G)			
3	Block sample 1			Phyllite					42.6	2.01	22.7	
4	Block sample 2			Phyllite					36.5	1.93	22.7	
5	Drill sample, 1.0 m			Phyllite					x	2.19	x	
6	Drill sample, 2.0 m			Phyllite					45.9	x	22.2	
7	Drill sample, 4.0 m			Phyllite					38.6	x	22	
8	Drill sample, 5.0 m			Phyllite					x	1.66	x	
9	Block sample 1			Quartzite					78.1	4.37	24	
10	Block sample 2			Quartzite					92.5	5.06	23	
11	Block sample 3			Quartzite					82.3	4.64	24.3	

### 5.2.1 Interpretation of geotechnical data & findings of core drillings

The exploratory core drilling works have been carried out using Rotary Core Drilling Rig for the landslides. The boreholes were logged continuously in the field. The borehole logs include visual classification of soil, no records of SPT/ DCPT, Monitoring the position of ground water table. The samples were collected from the exploration and carry out the necessary laboratory testing works.

Drill hole is done on the downhill slope of the road alignment up to 6.0 m depth, horizontal drilling have been perform in which boulders and sands are found at crown whereas from depth of 6.0 m, bedrocks of quartzite are found up to depth of exploration. The bedrocks are hard and fractured with less core recovery. No Ground water table was observed during the exploration. The prevalent test result are listed above which are considered for numerical modelling and analysis of slope instability.

### 5.3 Geophysical Investigation

Geophysical investigation done by using Micro tremor array measurement (MAM) survey has been carried out to explore both on the lithostratigraphic sequences, geometry of the landslide body (lateral extension and thickness), identifying the morethan 30 m depth from surfaces between the slide material and the underlying bedrock, thickness of landslide mass deposit at different locations of landslide area and bearing capacity of slope.

*Table 5-3: Interpretation of MAM of Tuin khola, landslide*

S.No.	Shear Wave Velocity (m/s)	Depth (m)	Type of Soil	Bearing Capacity (kN/m <sup>2</sup> )
1	443-481	6.50	Colluvial deposit with gravels and boulders	223-245
2	479-605	16.46	Highly weathered and fractured bedrock of phyllite with quartzite and limestone	239-302
3	605-1122	30.03	Fractured to competent bedrock of phyllite with quartzite and limestone.	302-589
4	1386-1521	>30.03	Competent bedrock of phyllite with quartzite and limestone.	589-897

The shear wave velocity profile shows that the detailed analysis of landslide area.

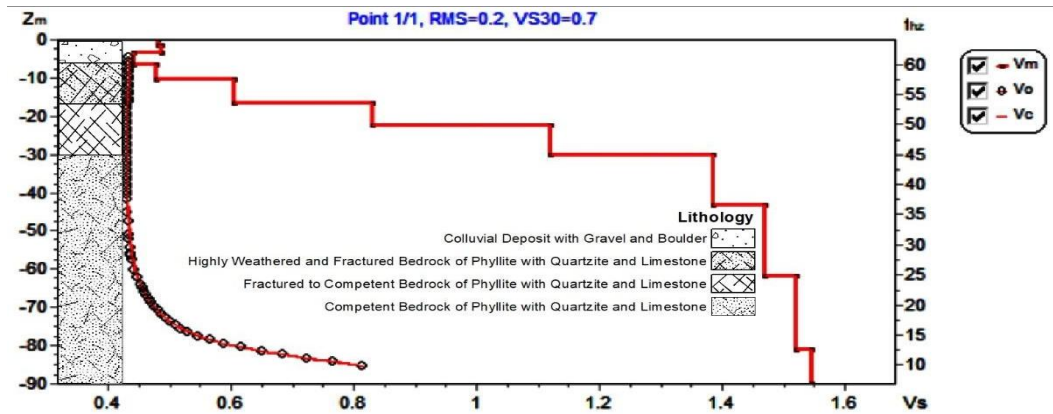


Figure 5-2: Shear wave velocity MAM test.

### 5.3.1 Interpretation of geophysical data

- Colluvial deposit with gravels and boulders (6.5 m)
    - Loose to medium dense soil, mixture of gravel and boulders.
    - Likely moderately compressible with lower strength.
  - Highly weathered and fractured bedrock (16.46 m)
    - The rock is heavily weathered; fractures reduce its strength.
    - May require careful stability consideration
  - Fractured to competent bedrock (30.03 m)
    - Increasing stiffness and load-bearing capacity with depth.
    - Suitable for foundations, though fractured zones may need treatment
  - Competent bedrock (>30.03 m)
    - Very strong, minimal deformation under load.
    - Ideal for deep foundations.
- a) Bearing capacity interpretation
- 6.5 m: 223–245 → Suitable for light to medium structures.
  - 16.46 m: 239–302 → can support moderate structures, but fractured rock may require foundation reinforcement.
  - 30.03 m: 302–589 → supports heavy structures; fractured rock may still need grouting or pile foundations.
  - 30.03 m: 589–897 → Ideal for very heavy foundations; minimal settlement expected.

## **5.4 Slope Stability Assessment**

The slope stability analysis is done with Morgenstern–Price method, which satisfies both force and moment equilibrium conditions. A slope is considered stable when the FS obtained from the Morgenstern–Price analysis exceeds the minimum acceptable limits.

For slope stability analysis, typical minimum acceptable FS values under seismic conditions, the minimum acceptable FS for excavated cut slopes is generally taken as 1.0, in accordance with standard engineering practice and relevant design guidelines.

## **5.5 Sub-Surface Conditions**

The sub-surface conditions of all the seven slides are determined based on the geophysical and drilling investigations carried out. The layers are briefly described as follows:

**Top Colluvium Layer:** At the crown of the landslide topsoil layer, there is colluvium deposits with silty sand, boulders and fractured rock fragments / gravels and boulders. This indicates that this area is composed of thick debris materials.

**Bottom Intact Bedrock:** Below the top colluvium deposit there is layer of dense overburden layer that is unsaturated landslide mass deposit with highly weathered and fractured bedrock of phyllite with quartzite and limestone sediments.

**Bedrock:** Weathered and fractured bed rock and competent bedrock is present at the lowest level below overburden mass. In some of the slides, the bedrock has been exposed. In our analysis, competent and weathered bedrock both are considered as bedrock with intermediate properties. This is fractured to competent bedrock of phyllite with quartzite and limestone.

## **5.6 Material Properties**

The geo-mechanical properties for bedrock are obtained using roc lab 1.0 software of conscience presented in Figure 5-3: Estimation of geo-mechanical properties of bedrock.

For geo-mechanical properties of the top colluvium deposit and bottom fine sediment deposit, results from the laboratory tests are used. The rock properties are calculated from rock lab1 and interpretation are tabulated in table.

Table 5-4: Estimation of geo-mechanical properties of bedrock

Rock Lab														
S. N	Type	UCS	GSI	mi	D	Slope Height	$\gamma$	$\sigma_3$	$E_m$	$m_b$	s	a	C	$\phi$
1	BMS	N/A	N/A	N/A	N/A	N/A	18.9						4.7	29.9
2	Phyllite (Weathered)	36.5	25	7	0.7	14.1	22	275.8	931.24	0.114	1.90E-05	0.531	57	34.58
2	Phyllite (Weathered)	36.5	25	7	0.7	23.12	22	410.3	931.24	0.114	1.90E-05	0.531	76	31.08
2	Phyllite (Weathered)	36.5	25	7	0.7	42.41	22	694.1	931.24	0.114	1.90E-05	0.531	108	26.9
2	Phyllite (Weathered)	36.5	25	7	0.7	68.44	22	1072.9	931.24	0.114	1.90E-05	0.531	142	23.76
2	Phyllite (Weathered)	36.5	25	7	0.7	90.92	22	1389.3	931.24	0.114	1.90E-05	0.531	168	21.98
3	Phyllite (Hard Rock)	45.9	35	7	0	33.04	24	771	2856.97	0.687	7.00E-04	0.516	269	46.17
3	Phyllite (Hard Rock)	45.9	35	7	0	55.87	24	1082.2	2856.97	0.687	7.00E-04	0.516	345	42.5
3	Phyllite (Hard Rock)	45.9	35	7	0	84.88	24	15.83.4	2856.97	0.687	7.00E-04	0.516	429	39.47
4	Quartzite (Weathered)	78.1	25	20	0.7	29.85	23	590.8	1362.29	0.325	1.90E-05	0.531	171	43.62
						55.13	23	1032.6	1362.29	0.325	1.90E-05	0.531	252	39.17
						83.73	23	1510.3	1362.29	0.325	1.90E-05	0.531	328	36.12
5	Quartzite(Hard Rock)	92.5	35	20	0	20.93	25	513.5	4055.75	1.963	7.00E-04	0.516	369	60.82
						42.3	25	974.2	4055.75	1.963	7.00E-04	0.516	534	56.79
						66	25	1460.4	4055.75	1.963	7.00E-04	0.516	690	53.99

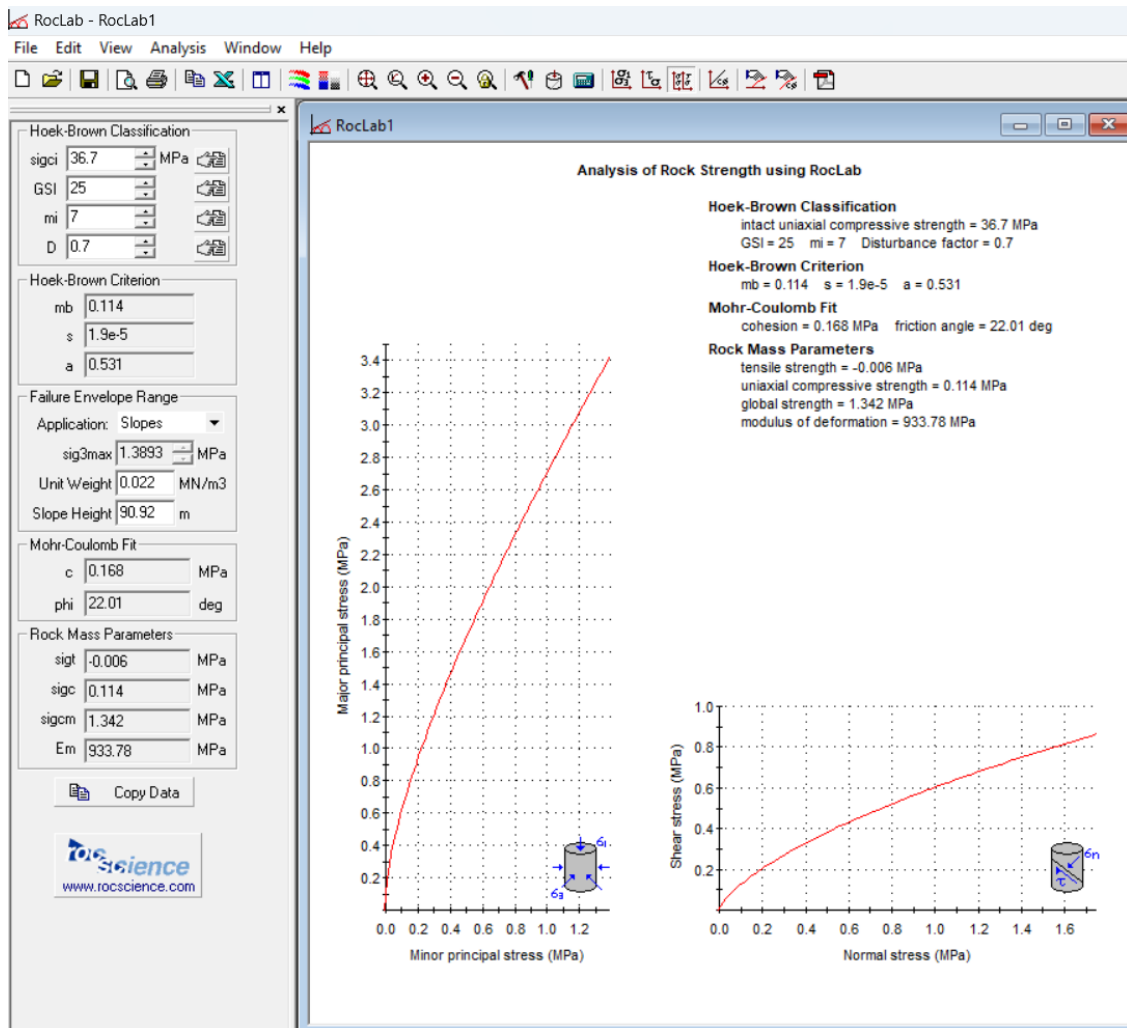


Figure 5-3: Estimation of geo-mechanical properties of bedrock.

## 5.7 Hydraulic Properties

The Hydraulic properties are assumed based on experiences and literatures, constant head permeability test and water recovery/loss condition during drilling. The defined hydraulic properties are also verified within the range.

## 5.8 Seismic Coefficient

Seismic Hazard Map is shown in figure 5-4 which state reduction factor of 50%, the calculated effective design seismic coefficient for this area. According to NBC105-2025, this study falls under Ichchhyakamana Gaunpalika of Chitwan district whose PGA factor is 0.35, so that the recommendation for the horizontal coefficient and vertical coefficient taken as half of the PGA value for slope stability analysis. This accounts for the fact that the entire sliding mass does not experience the peak acceleration simultaneously.

In mathematically,

$$k_h = 0.5 \times \text{PGA} = 0.175$$

Similarly,

The vertical seismic coefficient ( $k_v$ ) is often neglected (set to zero) in simplified analyses because the effect of vertical acceleration on stability is typically less significant than horizontal acceleration. However, when it is included, it is commonly taken as a fraction of ( $k_h$ ), usually between one-third and one-half of horizontal coefficient.

$$k_v = (0.33 \text{ to } 0.5) \times k_h = 0.12$$

Thus,  $k_h = 0.18$  for horizontal and  $k_v = 0.12$  for vertical.

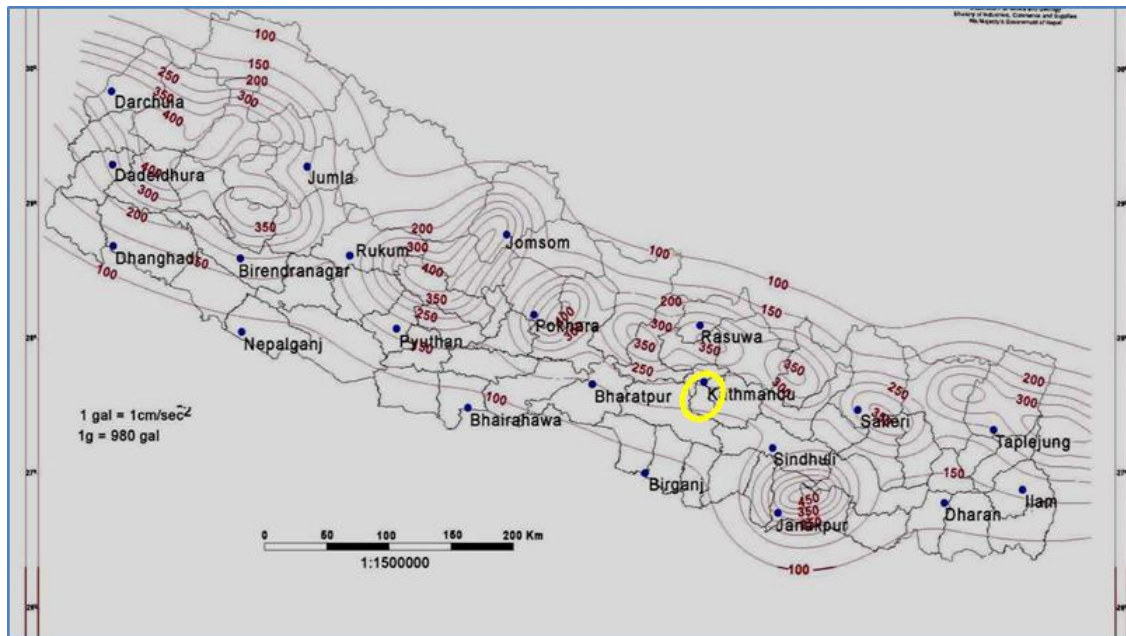


Figure 5-4: Probabilistic seismic hazard of Nepal (Source: DMG, 2002).

## 5.9 Slope Stability Analysis

The critical Slide is being chosen (Chainage 0+60) and slope profile are carried out for the slope stability analysis since colluvium deposits are observed during the field observation. The remaining Slide are not processed to stability analysis, since the slope observed are stable with rock outcrop exposed.

The best-fit slope stability model has been developed based on the interpretation of MAM results, borehole data, and the overall bedrock orientation. The slope stability analysis is carried out along critical MAM line selecting the most critical section

developing the profile from the developed topographic map of this Tuin khola, landslide.

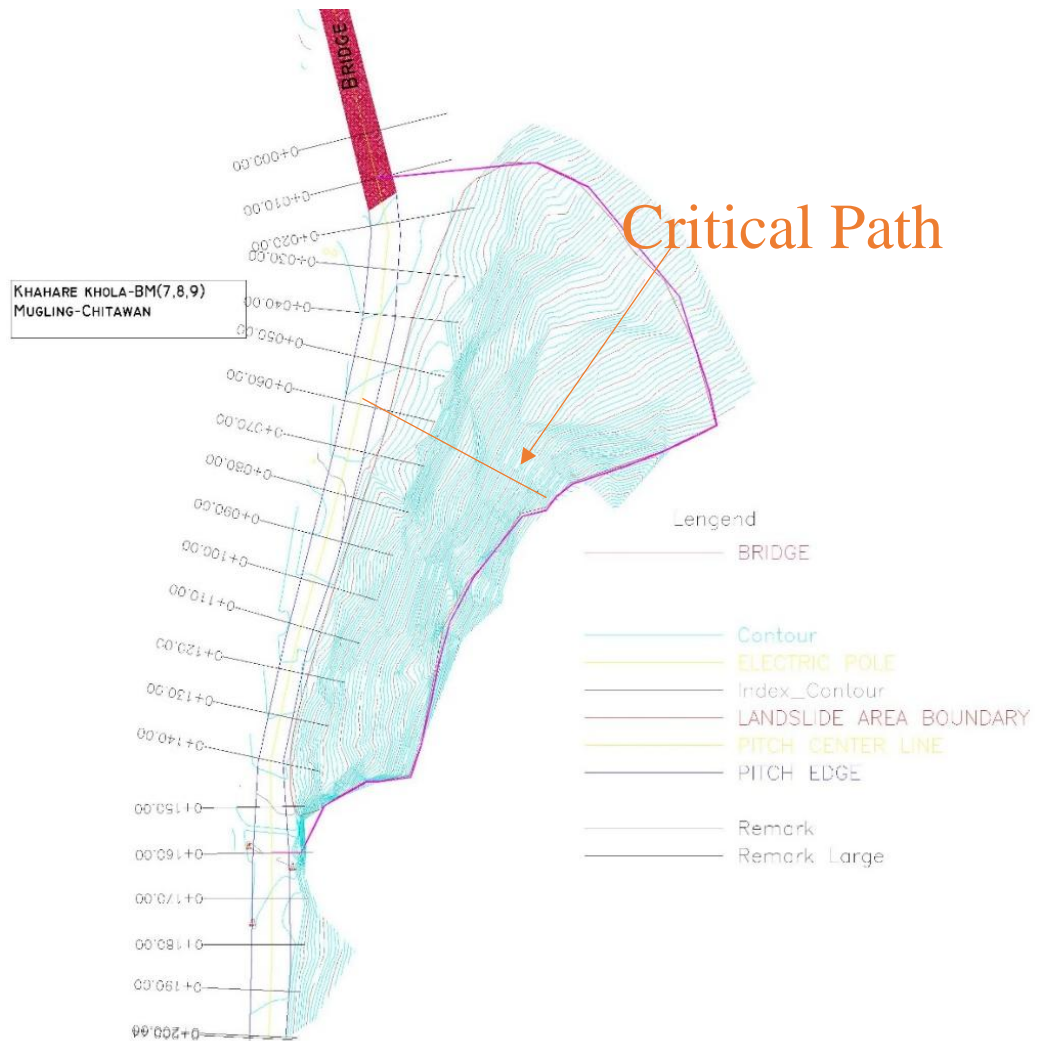
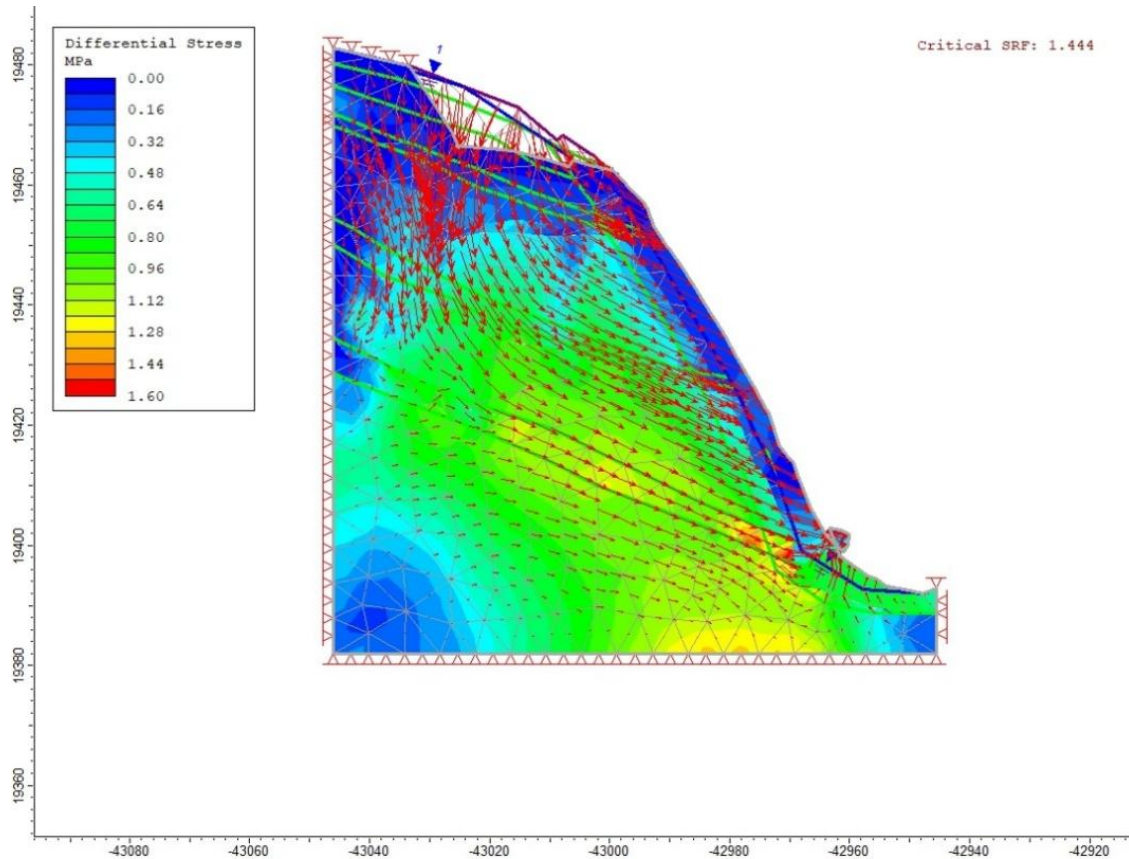


Figure 5-5: Contour map drawing of failure slope.

The slope stability analysis has been done based on the relation between the dipping of the natural hill slope and wedges formed by intersection of the foliation plane and joints. Location 3 predicted as critical and it revealed due to the occurrence of chances of wedge failure along the foliation plane. Wedges are formed by intersection of foliation (FP) and Joint (J3) is also critical because of chances of occurrence of central wedge failure. Remaining wedges seem to be stable and very less chances to occur the failure.

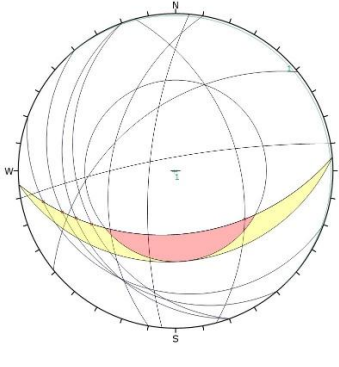
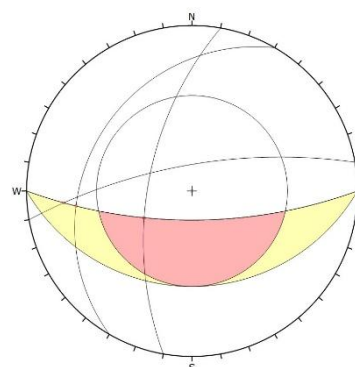
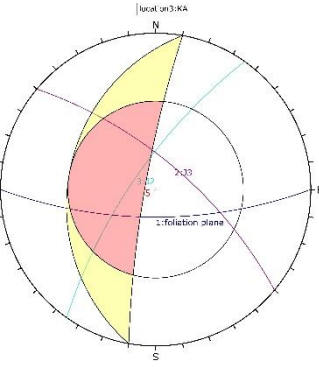
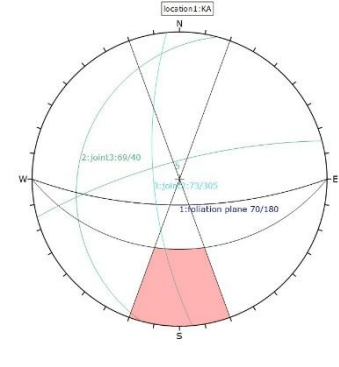
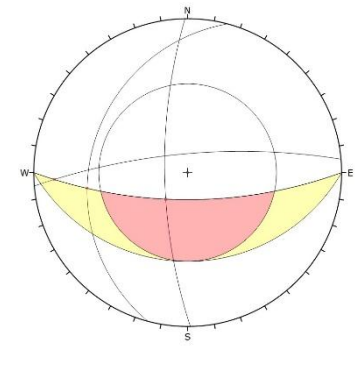


*Figure 5-6: Differential stress distribution and displacement vector.*

The deformation of slope profile is analyzed in Phase2 software as shown in figure above. The differential stress distribution and displacement vectors obtained from the analysis indicate the stress concentration within the slope mass. Higher stress zones are observed near the middle and toe portion of the slope, shown by yellow to red colors in the contour legend. The calculated critical Strength Reduction Factor (SRF) is 1.444, which indicates that the slope is marginally stable under the current conditions. The vector profile indicating the movement of the slope mass towards the toe region, highlighting the possibility of progressive slope deformation with unfavorable conditions such as heavy rainfall happen.

For each location kinematic analysis is done with dips software of rocscience. With help of it, calculate the probable failure. Movement takes place by sliding along both planes in the direction of line of intersection southwest dipping toward the road alignment, all other wedges failure seem to be stable because the wedges are formed just opposite to the dipping of the natural hill slope. So, there is less chances to occur the failure.

Table 5-5: Stereographic projection of rock mass exposed at slide area

		
<p>KA-1: 11.31 % Wedge Failure</p>	<p>KA-2: 50.0 % Wedge Failure</p>	<p>KA-3: 67.67 % Wedge Failure</p>
		
<p>KA-4: 23.33 % Flexural Toppling Failure</p>	<p>KA-5: 50.0 % Wedge Failure</p>	

## 5.10 Kinematic Analysis Result

This stereo net diagram presents a kinematic analysis for potential wedge sliding in a rock slope. The analysis indicates that a significant percentage of potential wedge failures are kinematically feasible and frictionally unstable under the given slope and discontinuity conditions Failure Mode Identified: Wedge Sliding

- The image shows that 66.67 % of the total area is critical for failure, specifically wedge sliding
- Remaining wedges seem to be stable and very less chances to occur the failure. The instabilities are structurally controlled and less

- The red shaded region lies inside the friction cone and within the daylight envelope → potential sliding wedge exists and other as so on.

### 5.11 Numerical Modelling with Slope/w

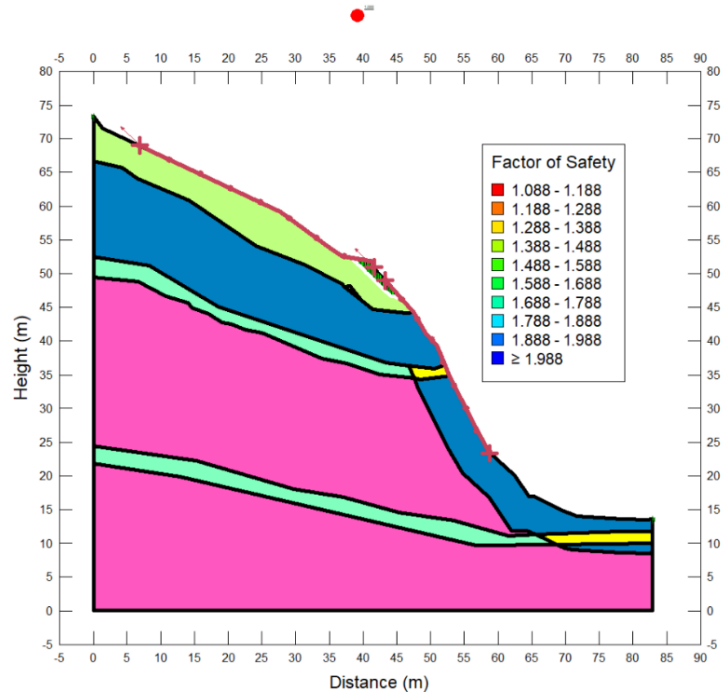


Figure 5-7: Stability analysis for the dry condition.

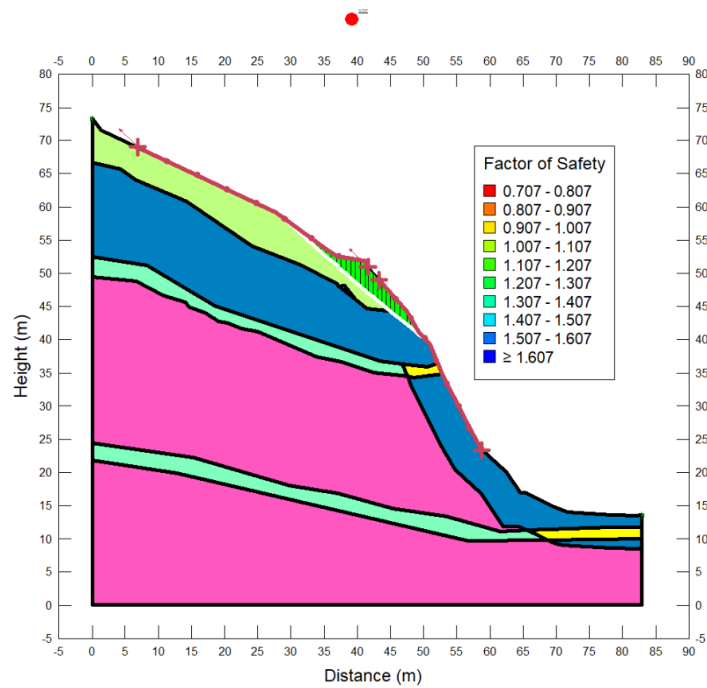


Figure 5-8: Stability analysis with fully saturated soil.

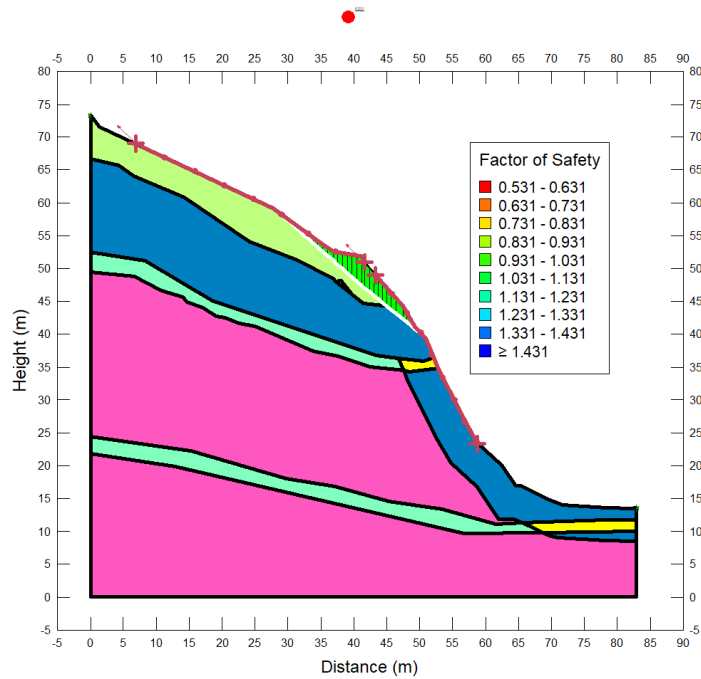


Figure 5-9: Stability analysis with fully saturated soil with seismic loading.

### 5.12 Groundwater and Stability Conditions

The slope stability analysis was carried out under three representative conditions to evaluate the performance of the slope under varying environmental and loading scenarios. The FS criteria were selected in accordance with standard engineering practice. Seismic Condition: Groundwater level assumed based on field observation or at the interface of the most vulnerable lithology, combined with seismic forces to represent critical conditions.

Table 5-6: FS comparison with case

S.No.	Case	Groundwater Conditions	FS
1	Dry Condition	No Presence of Ground water table (Static condition)	1.5
2	Wet Condition (monsoon)	Rock Slope: The saturated Gwt. within the tension crack or half depth of the slope saturated with groundwater along the active plane. Soil Slope: The Gwt. level is set at the interface of Boulder Mix Sand (BMS) and weathered Rock	1.2
3	Seismic Loading	Actual measurement or normal PGA level at the interface of most vulnerable lithologies	1.1

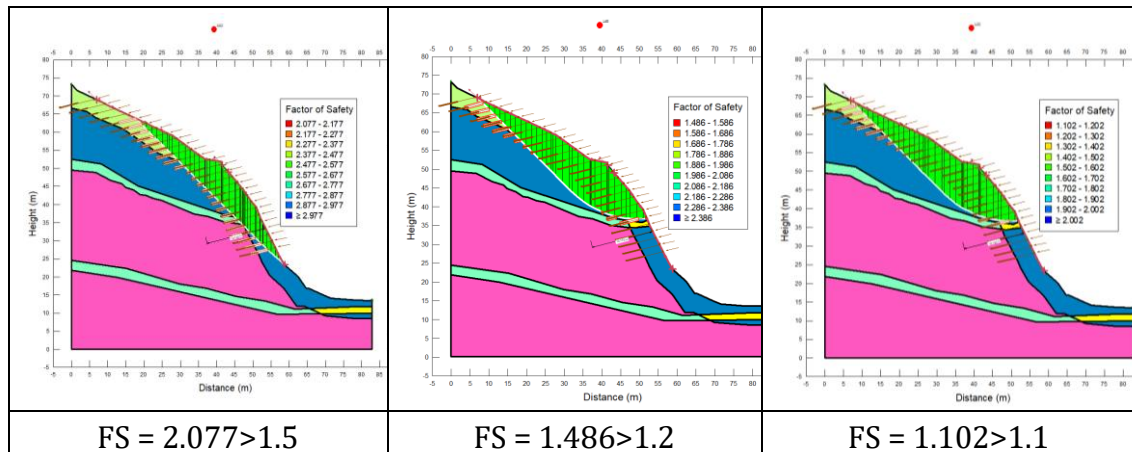


Figure 5-10: Circular failure analysis: Soil & weathered rock (Ch: 0+060).

The soil and weathered rock part at present cutting gradient is in stable state. But, considering the site Condition, due to highly fractured and thinly foliated highly weathered rock application of minimum reinforcement has been suggested in the weathered rock section.

### 5.13 Results of Slope Stability Analysis

The result of above data are analyzed and tabulated in Table 5-7, from the analysis, it is observed that the failure of the slope during rainfall intensity of  $3.5 \times 10^{-6}$  m/s. The critical slip circle passes from the rock line along with top and bottom overburden to the road section. Since the slide is critical at the road section, In general the FS are vigorously decreasing during the monsoon season, after channeling the water somewhat extend the FS. RCC cladding wall is necessary at the roadside slope for slope retaining purpose and reduce surface erosion.

Table 5-7: Results of slope stability analysis

Slope CH	Slope Conditions	Slope height	Factors of Safety			Remarks
			Dry	Rainfall	Seismic	
			1.5	1.2	1.10	
00+020	Intact rock and outer surface moderately weathered rock	23.46	2.718	2.616	1.921	Unstable
	Not required		-	-	-	Stable
00+040	Top portion containing colloidal soil, followed by weathered phyllite up to 5 m	59.71	1.265	0.674	0.495	Unstable

	and bed rock-phyllite with thin foliated band of quartzite					
	After Reinforce, rock bolt 25 mm dia, 2x1.5 m ctc, Length- 10 m		2.081	1.430	1.119	Satisfy all Stable condition
00+060	Top portion containing collovial soil, followed by weathered phyllite up to 5 m and bed rock-phyllite with thin foliated band of quartzite, weak and fracture zone.	92.5	1.465	0.863	0.691	Unstable
	After Reinforce, rock bolt 25 mm dia, 2x1.5 m ctc, Length- 10 m		2.077	1.486	1.102	Satisfy all Stable condition
00+100	Highly weathered, poor condition, steep slope. Chances of toppling failure.	64.82	1.115	0.471	0.501	Unstable
	After Reinforce, rock bolt 25 mm dia, 2x1.5 m ctc, Length- 7m		1.968	1.431	1.138	Satisfy all Stable condition
00+120	Highly weathered, poor condition, steep slope. Chances of wedge failure.	52.77	1.940	0.841	0.919	Unstable
	After Reinforce, Rock Bolt 25 mm dia, 2x1.5 m ctc, Length- 4 m		2.093	1.366	1.155	Rock bolt 4 m

### *Interpretation*

The Tuin Khola landslide is a large complex failure characterized by rock slide processes in the upper and central slopes, with debris-flow activity downslope. In

its central zone, where thick beds of stronger rocks like amphibolite and quartzite are exposed, the dominant failure is by block toppling, while on the upper slope, thinly foliated phyllite and slate exert their main controls through flexural and multiple rock topples. Unlike earlier Himalayan examples where strength contrast between mudstone and sandstone and simple river undercutting were considered the main triggers, the instability is instead more closely related to regional tectonic control by nearby thrust faults, with subsequent gravitational deformation, unfavorably oriented foliation and joint sets, intensely sheared and fractured zones, small faults within the slide mass, and toe undercutting; these all together govern the initiation and continued evolution of the massive landslide. The result of stability analysis is listed in table 5.7

#### 5.14 Analysis Results Validation with Phase2 (FEM).

The results obtained from Phase2 analysis, during the dry condition, saturated and seismic loading are shown in figure. It illustrate the development of critical stress and deformation zones within the 90 m high phyllite slope. Using the Strength Reduction Method (SRM), we can see that the slope is barely holding on even when dry, with its FS > 1. The data shows that deformation is mostly hitting the weakest spots: specifically the foliation planes which essentially dictate how the entire slope behaves.

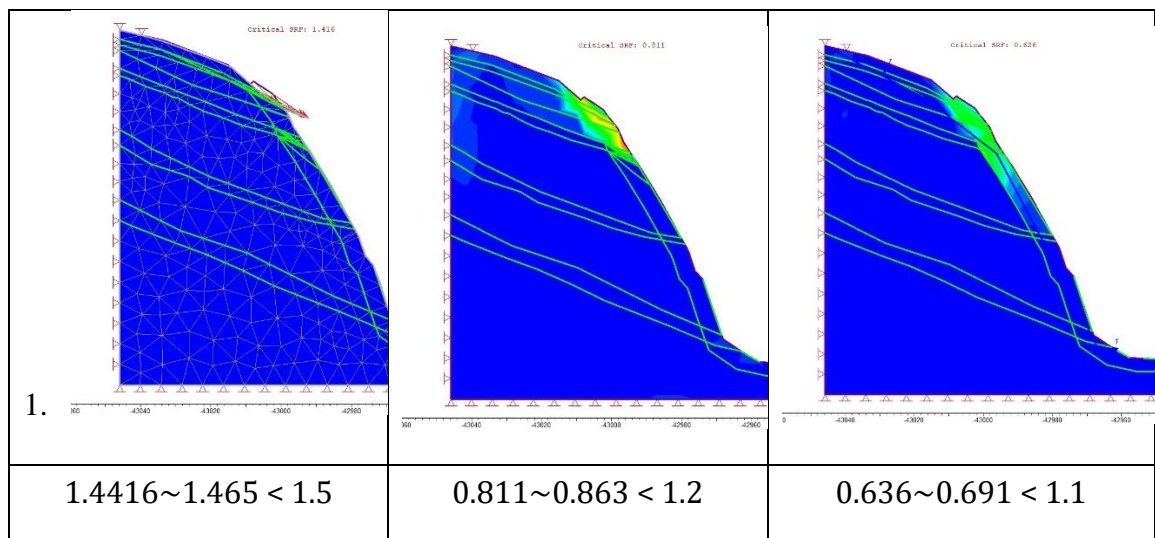


Figure 5-11: Comparison of result L3 for dry, saturated and seismic loading.

This figure 5.11 clearly shows a potential failure surface developing from the top of the slope down toward the toe. The way shear strain and deformation are distributed points to a progressive failure, with the slope mass pushing outward near the exposed face. When the slope becomes saturated, pore water pressure spikes, reducing effective stress and shear resistance. As a result, the failure zone

expands, pushing deeper into the slope and driving the FS below 1. This highlights a high risk of failure during heavy rainfall or extended wet periods. Ultimately, the analysis underscores that groundwater and structural weaknesses are the main drivers of instability here.

### 5.15 Protection Design Drawing of Slope Failure.

The effect of structural reinforcement rock bolts and subsurface drainage results in a stable slope configuration with a FS exceeding the recommended value (generally  $FS > 1.5$ ). This confirms that the proposed mitigation measures are adequate and efficient in controlling slope instability and ensuring long-term performance of the slope. Following are the design drawing.

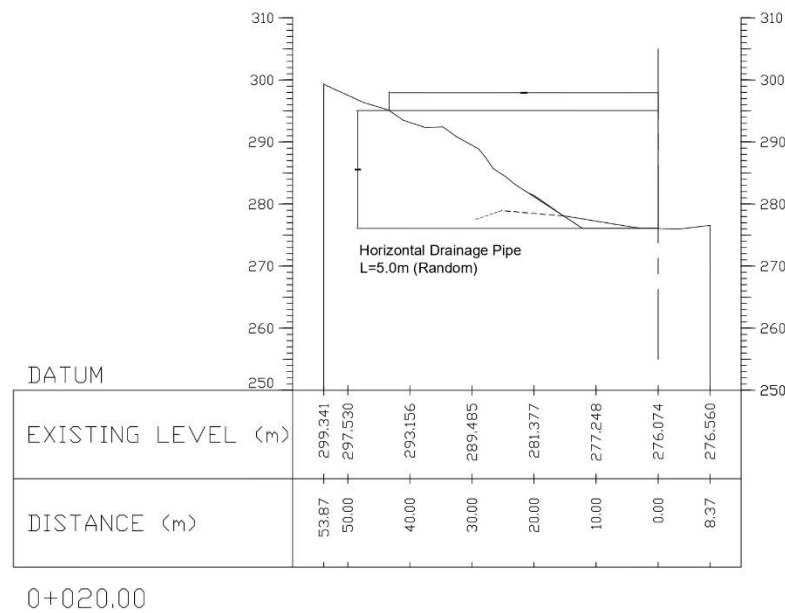


Figure 5-12: Protection of slope at Ch.: 00+020.



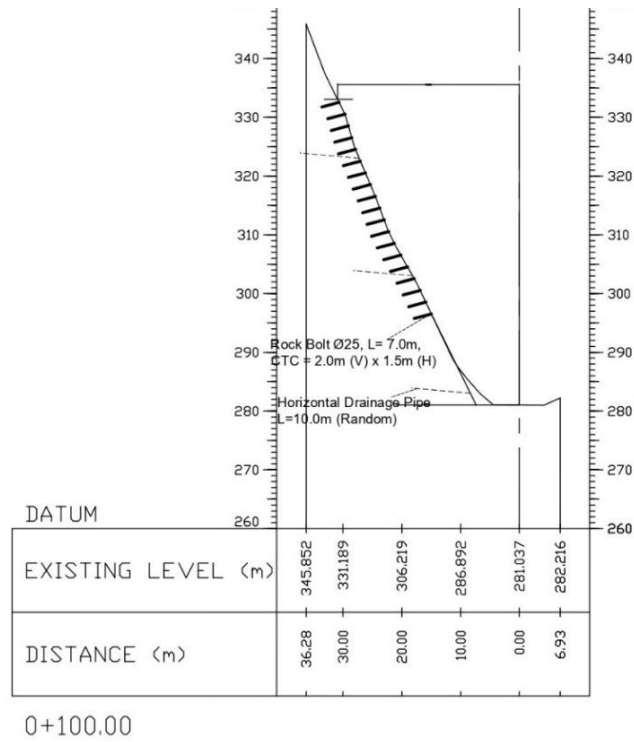


Figure 5-15: Protection of slope at Ch.: 00+100.

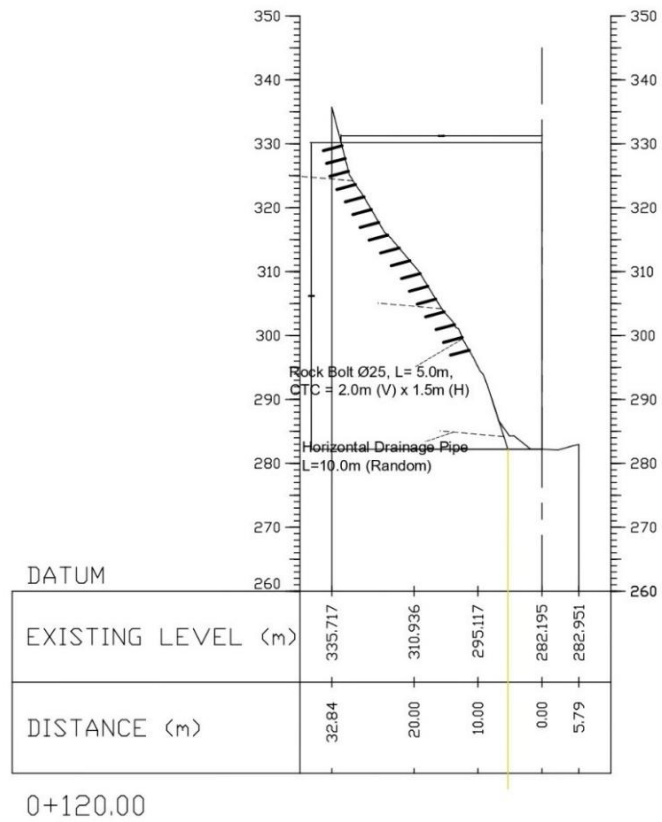


Figure 5-16: Protection of slope at Ch.: 00+120.

## 5.16 Results Verification

1. The Slope/w numerical results are considered valid as the computed factors of safety are consistent across LEM, agree with poor rock mass classification (RMR = 40, SMR = 24), and demonstrate realistic failure mechanisms supported by sensitivity analysis and field discontinuity data
2. Slope stability analysis by FEM namely Morgenstern price method, is done by using Slope/w software for the failure slope. The FS value calculated high as 0.717 and 0.696 as the lowvalue (Aakriti Koirala et al.,2024)
3. Generalised, limit equilibrium analyses, Morgenstern price method, rock slope stability, plane failure (Faridha Aprilia et al.,2024)
4. Rock Lab used for data validation which are drived from laboratory tests.
5. Phase2 software analysis of same slope under same loading condition with same material properties to match the result of Slope/w result that find the probably same

## 6. CONCLUSIONS AND RECOMMENDATIONS

### 6.1 Conclusions

The numerical analysis of the Tuin Khola slope (CH 00+020 to CH 00+120) indicates that the site is highly vulnerable to instability, particularly under rainfall conditions. The factor of safety varies significantly across sections, with several locations failing to meet the minimum stability requirements under both seismic and saturated conditions. The slope is composed of boulder mixed sand overlying highly weathered phyllite and fractured rock, with a weak shear zone governing instability. The analysis further demonstrates that appropriate stabilization measures, particularly rock bolting, can effectively enhance slope stability and ensure the long-term safety of the infrastructure. The key findings of the study are summarized as follows:

- The factor of safety (FS) ranges from 2.718 under dry conditions to 0.471 under rainfall, indicating critical instability in several sections.
- Most sections (CH 00+040 to CH 00+120) fail to satisfy minimum safety requirements under rainfall ( $FS \geq 1.2$ ) and seismic loading ( $FS \geq 1.1$ ).
- Rainfall-induced pore water pressure significantly reduces shear strength, causing up to 58% reduction in FS and triggering progressive deformation, confirming rainfall as the primary destabilizing factor.
- Installation of 25 mm diameter rock bolts (4–10 m length at 2 m × 1.5 m spacing) substantially improves slope stability and increases FS to acceptable levels.

Overall, the results demonstrate that moderate and cost-effective stabilization measures can significantly improve slope stability and ensure the long-term safety of the Tuin Khola slope and associated infrastructure.

## 6.2 Recommendations

Based on the geology, topography, subsurface condition and slope stability analysis, the following suitable mitigate measures are recommended:

- Scaling/removing the unstable mass, boulders existing on the slope.
- Clear the sliding mass accumulated in the hillside
- Provide 3-4 m RCC cladding wall at the hillside
- Pipe drains along the road at hillside
- Install a rock bolt with plate
- Cover the slope face with high tensile wire mesh

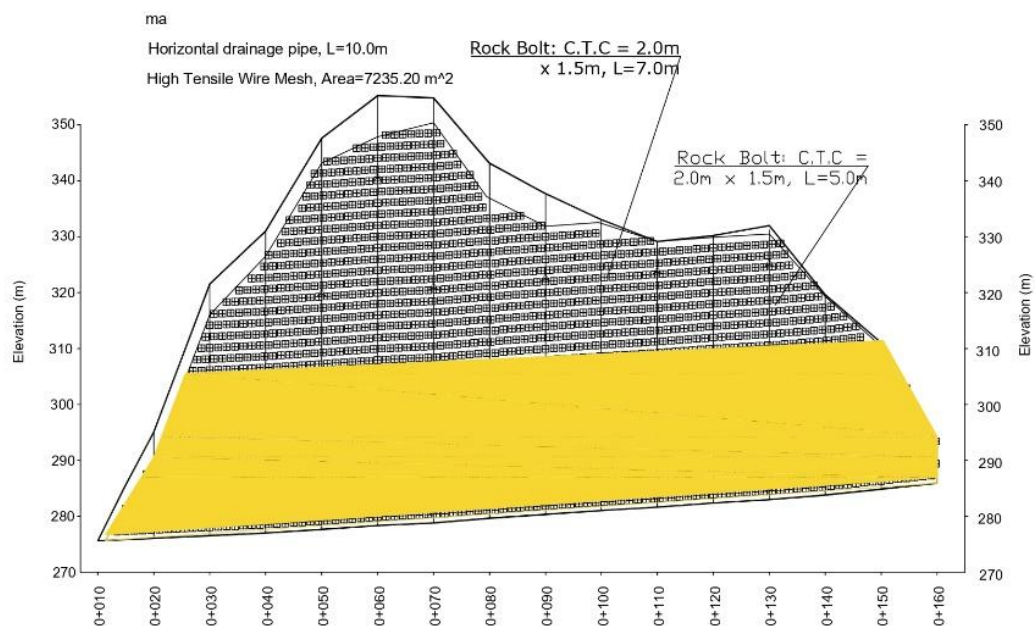


Figure 6-1: Layout of mitigate measures are recommended.

Variation of geotechnical parameters according to height and dip amount of slope and rainfall intensity is incorporated in this analysis. The geometrical parameters as taken as constant parameters. Hence, variation of geometry could be done in future work.

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## **ANNEX A: Geophysical, Geological, Geotechnical Analysis Data.**

**Pulchowk Campus**

Lalitpur.

**Test Result Summary Sheet**

Detailed Soil Investigation of Tuin khola.

S.No.	ISCS Classification	Sample No.	Percentage of			NMC	Specific Gravity	Direct shear test		Unit Weight		
			Gravel	Sand	Fines (Silt & Clay)			C	φ			
			%	%	%			GN/m <sup>2</sup>	Degree			
1	GP-SM	1	62.6	26.3	11.0	14.0	2.7	5.3	25.3	17.4		
	GP-SC	2	27.1	65.0	7.9	20.5	2.6	4.7	29.9	18.9		
	GP-SC	3	50.0	39.5	10.4	8.7	2.7	-	-	18.7		
	GP-SC	4	59.4	34.8	5.9	19.0	2.7	0.6	29.2	19.1		
2	GP	1	97.4	2.2	0.5	1.8	2.4	44.3	37.2	23.5		
	GP	2	95.2	4.5	0.2	1.4	2.4	51.4	38.8	24.2		
Rock sample	ISCS Classification	Pit of Test sample, m	Percentage of			NMC	Specific Gravity	Direct shear test		UCS	PI	Unit Weight
			Gravel	Sand	Fines (Silt & Clay)			C	φ			
			%	%	%			GN/m <sup>2</sup>	Degree			
3	Block sample 1		Phyllite							42.6	2.01	22.7
4	Block sample 2		Phyllite							36.5	1.93	22.7
5	Drill sample, 1.0 m		Phyllite							x	2.19	x
6	Drill sample, 2.0 m		Phyllite							45.9	x	22.2
7	Drill sample, 4.0 m		Phyllite							38.6	x	22
8	Drill sample, 5.0 m		Phyllite							x	1.66	x
9	Block sample 1		Quartzite							78.1	4.37	24
10	Block sample 2		Quartzite							92.5	5.06	23
11	Block sample 3		Quartzite							82.3	4.64	24.3

## Pulchowk Campus

Lalitpur.

### Detailed Soil Investigation of Tuin khola.

Initial Weight:		2116				
Bore hole :		1			Gravel	
Wt. of Sample before wash (gms):		2,116.00			62.64	
Wt. of Sample after wash (gms):		1,882.50			Sand	
Depth (m):		S-1			11.03	
Seive Size (mm)	Wt. of Retained soil (gm)	Cumulative weight Retained (gm)	Cumulative percentage retined %	Percent passing %	<div style="text-align: center;"> <b>Particle Size Distribution Curve</b> </div>	
49	650.00	650.00	30.72	69.28		
37.5	191.83	841.83	39.78	60.22		
25	169.13	1010.96	47.78	52.22		
19	127.44	1138.40	53.80	46.20		
6.3	63.73	1202.13	56.81	43.19		
4.75	123.32	1325.45	62.64	37.36		
2.36	101.98	1427.43	67.46	32.54		
2	98.60	1526.03	72.12	27.88		
1.18	27.35	1553.38	73.41	26.59		
1	57.64	1611.02	76.14	23.86		
0.85	42.09	1653.11	78.12	21.88		
0.6	40.63	1693.74	80.04	19.96		
0.425	90.05	1783.79	84.30	15.70		
0.3	27.51	1811.30	85.60	14.40		
0.25	13.26	1824.56	86.23	13.77		
0.15	29.98	1854.54	87.64	12.36		
0.075	27.96	1882.50	88.97	11.03		
Pan	233.50	2116.00	100.00	0.00		
	2116					

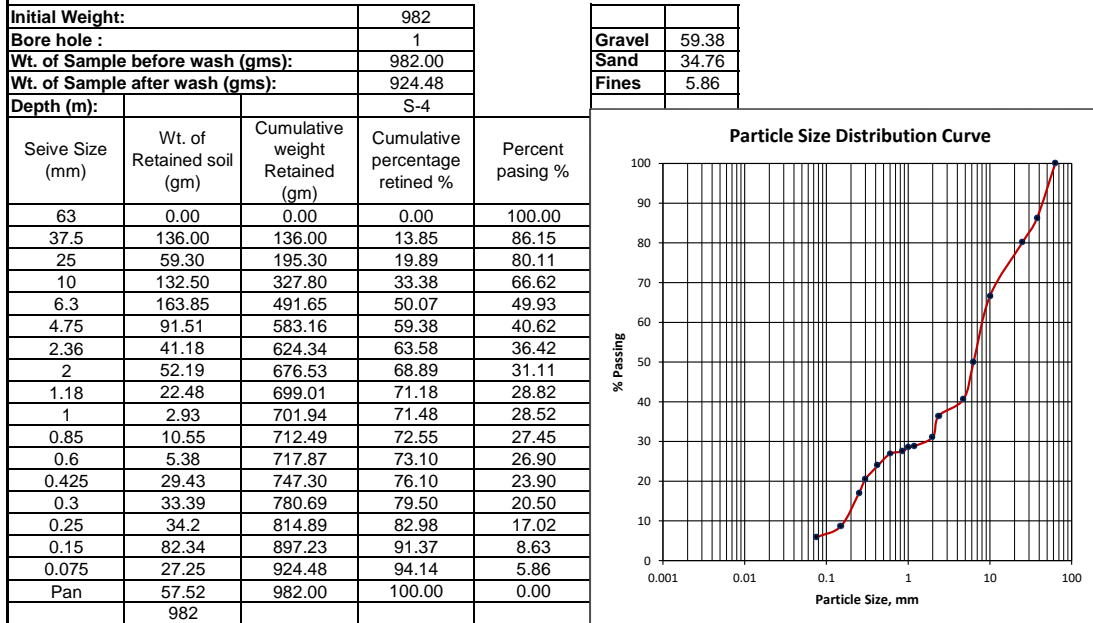
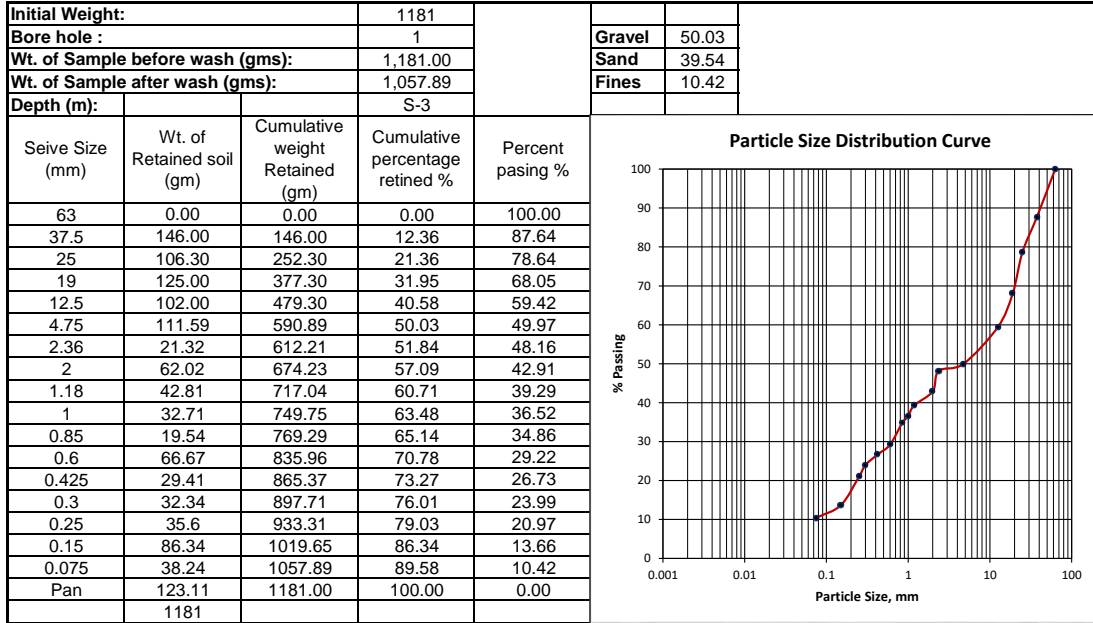
  

Initial Weight:		1592				
Bore hole :		1			Gravel	
Wt. of Sample before wash (gms):		1,592.00			27.13	
Wt. of Sample after wash (gms):		1,466.29			Sand	
Depth (m):		S-2			7.90	
Seive Size (mm)	Wt. of Retained soil (gm)	Cumulative weight Retained (gm)	Cumulative percentage retined %	Percent passing %	<div style="text-align: center;"> <b>Particle Size Distribution Curve</b> </div>	
37.5	0.00	0.00	0.00	100.00		
25	113.06	113.06	7.10	92.90		
19	97.26	210.32	13.21	86.79		
10	126.30	336.62	21.14	78.86		
6.3	54.80	391.42	24.59	75.41		
4.75	40.55	431.97	27.13	72.87		
2.36	28.63	460.60	28.93	71.07		
2	49.63	510.23	32.05	67.95		
1.18	76.32	586.55	36.84	63.16		
1	163	749.55	47.08	52.92		
0.85	2.59	752.14	47.24	52.76		
0.6	221.3	973.44	61.15	38.85		
0.425	291	1264.44	79.42	20.58		
0.3	77.2	1341.64	84.27	15.73		
0.25	31.63	1373.27	86.26	13.74		
0.15	51.7	1424.97	89.51	10.49		
0.075	41.32	1466.29	92.10	7.90		
Pan	125.71	1592.00	100.00	0.00		
	1592					

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### Detailed Soil Investigation of Tuin khola.



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**Detailed Soil Investigation of Tuin khola.**

<b>Initial Weight:</b>		50365				
<b>Bore hole :</b>		2			<b>Gravel</b> 97.38	
<b>Wt. of Sample before wash</b>		50,365.00			<b>Sand</b> 2.17	
<b>Wt. of Sample after wash (gms)</b>		50,136.41			<b>Fines</b> 0.45	
<b>Depth (m):</b>		S-6				
<b>Seive Size (mm)</b>	<b>Wt. of Retained soil (gm)</b>	<b>Cumulative weight Retained (gm)</b>	<b>Cumulative percentage retained %</b>	<b>Percent passing %</b>		
300	0.00	0.00	0.00	100.00		
150	16500.00	16500.00	32.76	67.24		
80	12806.00	29306.00	58.19	41.81		
63	12125.00	41431.00	82.26	17.74		
49	5502.00	46933.00	93.19	6.81		
37.5	2111.00	49044.00	97.38	2.62		
4.75	16.50	49060.50	97.41	2.59		
2	8.25	49068.75	97.43	2.57		
1.18	12.69	49081.44	97.45	2.55		
1	2.84	49084.28	97.46	2.54		
0.85	8.98	49093.26	97.47	2.53		
0.6	78.5	49171.76	97.63	2.37		
0.425	35.69	49207.45	97.70	2.30		
0.3	250	49457.45	98.20	1.80		
0.25	385	49842.45	98.96	1.04		
0.15	285	50127.45	99.53	0.47		
0.075	8.96	50136.41	99.55	0.45		
Pan	228.59	50365.00	100.00	0.00		
	50365					

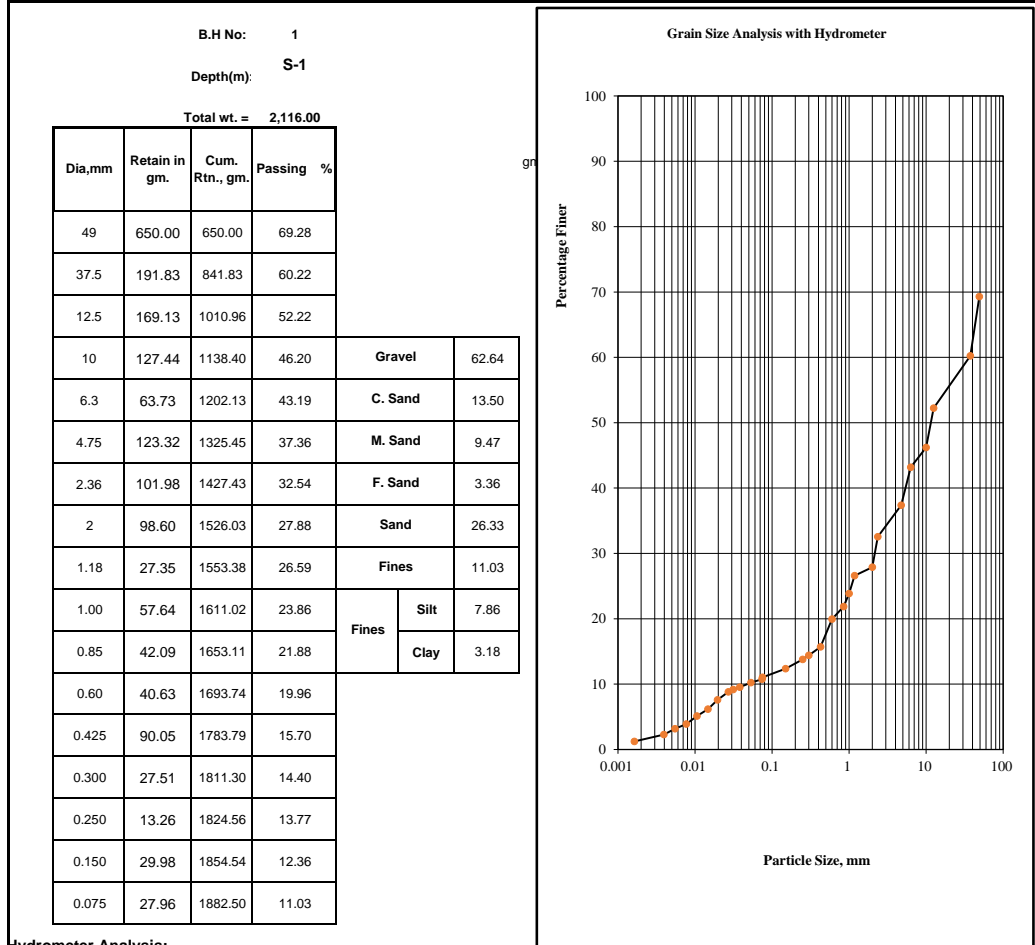
<b>Initial Weight:</b>		48952				
<b>Bore hole :</b>		1			<b>Gravel</b> 95.23	
<b>Wt. of Sample before wash</b>		48,952.00			<b>Sand</b> 4.55	
<b>Wt. of Sample after wash (gms)</b>		48,843.54			<b>Fines</b> 0.22	
<b>Depth (m):</b>		S-7				
<b>Seive Size (mm)</b>	<b>Wt. of Retained soil (gm)</b>	<b>Cumulative weight Retained (gm)</b>	<b>Cumulative percentage retained %</b>	<b>Percent passing %</b>		
300	0.00	0.00	0.00	100.00		
150	14695.00	14695.00	30.02	69.98		
80	10856.00	25551.00	52.20	47.80		
63	17853.00	43404.00	88.67	11.33		
49	2245.00	45649.00	93.25	6.75		
37.5	968.00	46617.00	95.23	4.77		
4.75	129.60	46746.60	95.49	4.51		
2	745.60	47492.20	97.02	2.98		
1.18	122.20	47614.40	97.27	2.73		
1	12.85	47627.25	97.29	2.71		
0.85	74.63	47701.88	97.45	2.55		
0.6	84.6	47786.48	97.62	2.38		
0.425	123.6	47910.08	97.87	2.13		
0.3	425.8	48335.88	98.74	1.26		
0.25	369.5	48705.38	99.50	0.50		
0.15	89.2	48794.58	99.68	0.32		
0.075	48.96	48843.54	99.78	0.22		
Pan	108.46	48952.00	100.00	0.00		
	48952					

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**Sieve Analysis & Hydrometer**

Detailed Soil Investigation of Tuin khola.



**Hydrometer Analysis:-**

Hydrometer type :	Gs of solids : 2.658	rw = 0.9978
Dispersing agent : (NaPO3) <sub>6</sub>	5 gm Amount :	Wt. of soil, Ws = 50.00 gms.
Zero correctio -0.00425	Meniscus correction : 0.5	Passing by .075mm= 11.03 %

Elapsed time (t) minute	Temp °C	Actual Hydro. reading	Corrected Hydrometer Reading	Hyd. Reading in water $R_w$ $\frac{R_w}{100(rw-1)}$	R - R <sub>w</sub>	K1	% Finer $K1 \cdot (R - R_w)$	Hyd. Corr. for meniscus & Temp R	Effective length L cm	L/t	K2	Diameter (D), mm $K2 \cdot \text{Sqrt}(L/t)$	Actual % Finer
0.50	14	1.0325	28.2500	-2.25	30.50	3.197	97.50	26.23	11.993	23.99	0.0152	0.0743	10.76
1	14	1.0310	26.7500	-2.25	29.00	3.197	92.71	24.73	12.239	12.24	0.0152	0.0531	10.23
2	14	1.0290	24.7500	-2.25	27.00	3.197	86.31	22.73	12.567	6.28	0.0152	0.0380	9.52
3	14	1.0280	23.7500	-2.25	26.00	3.197	83.12	21.73	12.731	4.24	0.0152	0.0312	9.17
4	14	1.0270	22.7500	-2.25	25.00	3.197	79.92	20.73	12.895	3.22	0.0151	0.0271	8.82
8	14	1.0235	19.2500	-2.25	21.50	3.197	68.73	17.23	13.469	1.68	0.0151	0.0196	7.58
15	14	1.0195	15.2500	-2.25	17.50	3.197	55.94	13.23	14.125	0.94	0.0152	0.0147	6.17
30	13	1.0165	12.2500	-2.25	14.50	3.197	46.35	9.87	14.676	0.49	0.0152	0.0106	5.12
60	13	1.013	8.7500	-2.25	11.00	3.198	35.17	6.37	15.250	0.25	0.0153	0.0077	3.88
120	12	1.011	6.7500	-2.25	9.00	3.198	28.78	4.01	15.637	0.13	0.0153	0.0055	3.18
240	12	1.0085	4.2500	-2.25	6.50	3.198	20.78	1.51	16.047	0.07	0.0153	0.0039	2.29
1440	12	1.0055	1.2500	-2.25	3.50	3.198	11.19	-1.49	16.539	0.01	0.0153	0.0016	1.23

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**NATURAL MOISTURE CONTENT**

Detailed Soil Investigation of Tuin khola.

Sample No.	point	Wt. of Cont. + Wet Soil (gm)	Wt. of Cont. + Dry Soil (gm)	Wt. of Water (gm)	Wt. of Empty Container (gm)	Wt. of Dry Soil (gm)	Moisture Content (%)
<b>1</b>							
SN -1	1.0	748.14	668.92	79.22	104.00	564.92	14.02
SN -2	2.0	1402.68	1199.98	202.70	213.00	986.98	20.54
SN -3	3.0	1402.34	1301.00	101.34	130.00	1171.00	8.65
SN -4	4.0	2152.00	1840.00	312.00	201.00	1639.00	19.04
SN -5	5.0	2901.66	2855.60	46.06	272.00	2583.60	1.78
SN -6	6.0	3651.32	3605.00	46.32	343.00	3262.00	1.42

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**SPECIFIC GRAVITY (IS: 2720 ( Part III/Set 1 ) - 1980)**

Detailed Soil Investigation of Tuin khola.

**Specific gravity of soil**

	Determination Number	point	Pycnometer No	Temp. of water, °C	Wt. of Pyc, W <sub>1</sub> (g)	Wt. of Pyc + Dry soil, W <sub>2</sub> (g)	Wt. of Pyc + Soil + Water, W <sub>3</sub> (g)	Wt. of Pyc + Water, W <sub>4</sub> (g)	Wt. of Dry Soil, W (g)	Sp.Gravit <sub>y</sub> = $\frac{W_d/W_d - (W_3 - W_4)}{(W_3 - W_4)}$	Average Sp.Gravit <sub>y</sub>	K= Relative density of water at room temperature/ Relative density of water at 27°C	Specific Gravity (G) at 27 deg C
	1	1.00	5	19	71.64	91.76	183.89	171.34	20.12	2.66	2.66	1.00	2.66
	2		6	19	70.12	100.13	188.49	169.78	30.01	2.66			
	1	2.00	1	20	77.21	98.2	189.98	176.91	20.99	2.65	2.65	1.00	2.65
	2		2	20	78.88	108.31	196.59	178.3	29.43	2.64			
	1	3.00	3	20	71.31	91.17	183.33	170.92	19.86	2.67	2.66	1.00	2.66
	2		4	20	72.28	101.63	189.89	171.63	29.35	2.65			
	1	4.00	1	20	77.2	97.21	183.86	171.34	20.01	2.67	2.67	1.00	2.67
	2		2	20	78.88	108.43	196.79	178.3	29.55	2.67			
2	1	1.00	1	22	569	1088.6	1488.6	1195.6	519.6	2.29	2.40	1.00	2.40
	2		2	22	598	1078.6	1578.6	1290.3	480.6	2.50			
	1	2.00	1	22	569	1097.6	1587.6	1290.3	528.6	2.29	2.36	1.00	2.36
	2		2	22	598	1028.7	1448.7	1195.6	430.7	2.43			

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**COMPACTION TEST**  
(As per IS 2720 Part-7 & 8)

Detailed Soil Investigation of Tuin khola.							
Location	twin khola, narayanghat mugling Road						
Type of compaction:	Heavy					wt. of rammer:	4.89 kg
No. of layers:	5 layer					No. of blows/layer:	56 blows
Test Data							
<b>Compaction:</b>							Remarks
Determination No.			1	2	3	4	5
Mass of empty mould + base plate	Kg	6.148	6.148	6.148	6.148	6.148	
Mass of mould + base plate + compacted soil	Kg	10.470	10.710	10.880	10.690	10.620	
mass of compacted soil	Kg	4.322	4.562	4.732	4.542	4.472	
Volume of mould	cc	2250.0	2250.0	2250.0	2250.0	2250.0	
Bulk Density	gm/cc	1.921	2.028	2.103	2.019	1.988	
Dry Density	gm/cc	<b>1.728</b>	<b>1.815</b>	<b>1.855</b>	<b>1.763</b>	<b>1.729</b>	
<b>Moisture content:</b>							
Container No.							
Empty weight of Container	kg	28.3	31.2	29.64	30.24	30.65	
Wt. of container and Wet Soil	kg	58.29	66.32	61.28	62.21	64.22	
Wt. of container and Dry Soil	kg	55.28	62.64	57.55	58.16	59.85	
Mass of water	kg	3.01	3.68	3.73	4.05	4.37	
Mass of dry soil	kg	26.98	31.44	27.91	27.92	29.2	
Water content		0.1116	0.1170	0.1336	0.1451	0.1497	
Water content (%)	%	<b>11.16</b>	<b>11.70</b>	<b>13.36</b>	<b>14.51</b>	<b>14.97</b>	
<p style="text-align: center;"><b>OMC Vs MDD</b></p> <p style="text-align: center;">Dry density (gm/cc)</p> <p style="text-align: center;">Moisture Content (%)</p>							
MDD (g/cc):-	<b>1.855</b>						
OMC (%):-	<b>13.36</b>						

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### ATTERBERG'S LIMIT

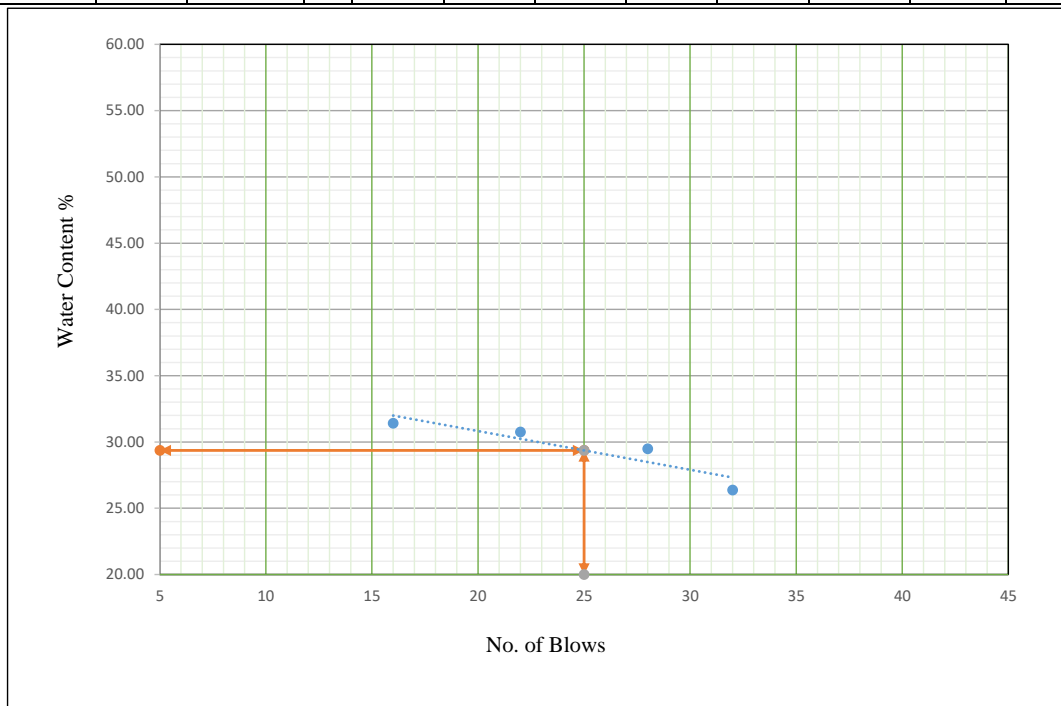
(As per IS 2720 Part-5)

Detailed Soil Investigation of Tuin khola.

Tested By : Aayush yadav

Depth            1  
B.H. No.        S-1

		Liquid Limit			
Determination No.		20	17	22	18
Empty weight of Container	(g)	15.76	16.18	15.67	15.66
Wt. of container and Wet Soil	(g)	30.9	33.44	31.23	29.63
Wt. of container and Dry Soil	(g)	27.74	29.51	27.57	26.29
Weight of Dry Soil	(g)	11.98	13.33	11.9	10.63
Moisture Content	(%)	26.38	29.48	30.76	31.42
No. of Blows	N	32	28	22	16



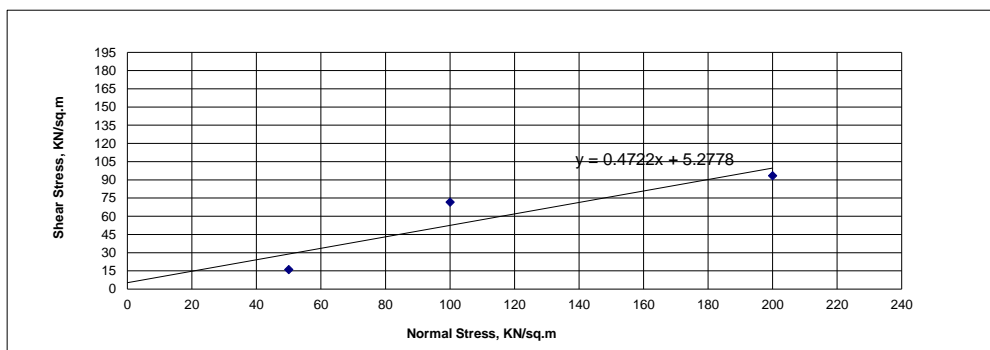
Liquid Limit LL:        29.36  
 Plastic Limit PL:       -  
 Plasticity Index PI:    -

**Pulchowk Campus**  
Lalitpur.

**Direct Shear Test (IS : 2720 ( Part 13 ) - 1986)**

**Detailed Soil Investigation of Tuin khola.**

Bore Hole No:	1					
Sample No.:					PRG Factor, kN/Div.:	0.002
Depth (m)	S-1				Area m <sup>2</sup> :	0.0036
Test No. 1	1		2		3	
Wt. of Mould + sample (gm)	266.96					
Wt. of mould (gm)	107.09					
Density (gm/cm <sup>3</sup> )	1.78					
Weight of cont. + wet soil (gm)	119.00					
Weight of cont. + dry soil (gm)	106.23					
Wt of cont.(gm)	30.65					
Wt of dry soil(gm)	75.58					
Wt of water (gm)	12.77					
M/C, % :	16.90					
Horiz. Dial Reading 0.01mm) (x	Normal Load 50 kN/m <sup>2</sup>		Normal Load 100.0 kN/m <sup>2</sup>		Normal Load 200.0 kN/m <sup>2</sup>	
	Load Ring Dial Reading	Shear Stress KN/m <sup>2</sup>	Load Ring Dial Reading	Shear Stress KN/m <sup>2</sup>	Load Ring Dial Reading	Shear Stress KN/m <sup>2</sup>
0.00	0.00	0.00	0.00	0.00	0.00	0.00
25.00	3.00	1.67	8.00	4.44	12.00	6.67
50.00	5.00	2.78	11.00	6.11	14.00	7.78
100.00	8.00	4.44	17.00	9.44	19.00	10.56
125.00	15.00	8.33	23.00	12.78	26.00	14.44
150.00	19.00	10.56	31.00	17.22	34.00	18.89
175.00	24.00	13.33	42.00	23.33	41.00	22.78
200.00	29.00	16.11	50.00	27.78	48.00	26.67
250.00			58.00	32.22	57.00	31.67
300.00			67.00	37.22	66.00	36.67
350.00			76.00	42.22	75.00	41.67
400.00			82.00	45.56	84.00	46.67
450.00			90.00	50.00	93.00	51.67
500.00			98.00	54.44	105.00	58.33
550.00			106.00	58.89	119.00	66.11
600.00			118.00	65.56	134.00	74.44
700.00			129.00	71.67	149.00	82.78
800.00					168.00	93.33
900.00						
1000.00						
1100.00						
1200.00						



C            5.28      KN/m<sup>2</sup>  
 Φ            25.28      Degree

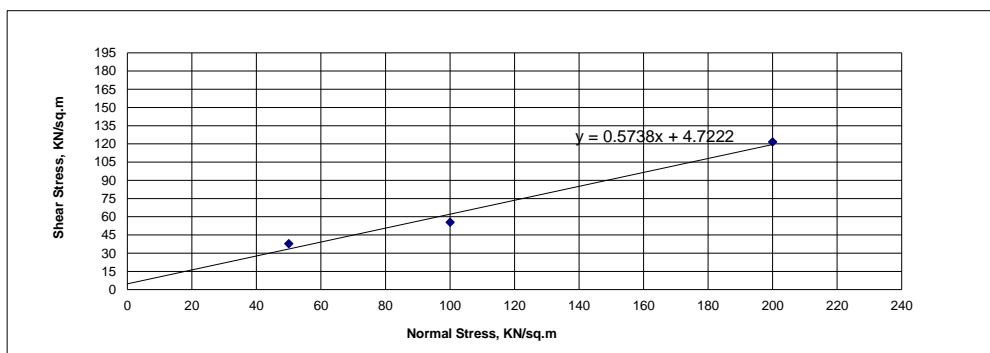
**Pulchowk Campus**

Lalitpur.

**Direct Shear Test (IS : 2720 ( Part 13 ) - 1986)**

**Detailed Soil Investigation of Tuin khola.**

Bore Hole No:	1					
Sample No.:					PRG Factor, kN/Div.:	0.002
Depth (m)	S-2				Area m <sup>2</sup> :	0.0036
Test No. 1	1		2		3	
Wt. of Mould + sample (gm)	267.62					
Wt. of mould (gm)	107.09					
Density (gm/cm <sup>3</sup> )	1.78					
Weight of cont. + wet soil (gm)	132.12					
Weight of cont. + dry soil (gm)	117.32					
Wt of cont.(gm)	32.21					
Wt of dry soil(gm)	85.11					
Wt of water (gm)	14.80					
M/C, % :	17.39					
Horiz. Dial Reading (x 0.01mm)	Normal Load 50 kN/m <sup>2</sup>		Normal Load 100.0 kN/m <sup>2</sup>		Normal Load 200.0 kN/m <sup>2</sup>	
	Load Ring Dial Reading	Shear Stress KN/m <sup>2</sup>	Load Ring Dial Reading	Shear Stress KN/m <sup>2</sup>	Load Ring Dial Reading	Shear Stress KN/m <sup>2</sup>
0.00	0.00	0.00	0.00	0.00	0.00	0.00
25.00	5.00	2.78	7.00	3.89	10.00	5.56
50.00	9.00	5.00	14.00	7.78	19.00	10.56
100.00	16.00	8.89	23.00	12.78	28.00	15.56
125.00	17.00	9.44	30.00	16.67	35.00	19.44
150.00	24.00	13.33	38.00	21.11	42.00	23.33
175.00	28.00	15.56	47.00	26.11	50.00	27.78
200.00	37.00	20.56	55.00	30.56	59.00	32.78
250.00	39.00	21.67	62.00	34.44	67.00	37.22
300.00	48.00	26.67	69.00	38.33	76.00	42.22
350.00	47.00	26.11	75.00	41.67	93.00	51.67
400.00	59.00	32.78	80.00	44.44	105.00	58.33
450.00	68.00	37.78	87.00	48.33	118.00	65.56
500.00			94.00	52.22	132.00	73.33
550.00			100.00	55.56	153.00	85.00
600.00					179.00	99.44
700.00					197.00	109.44
800.00					219.00	121.67
900.00						
1000.00						
1100.00						
1200.00						



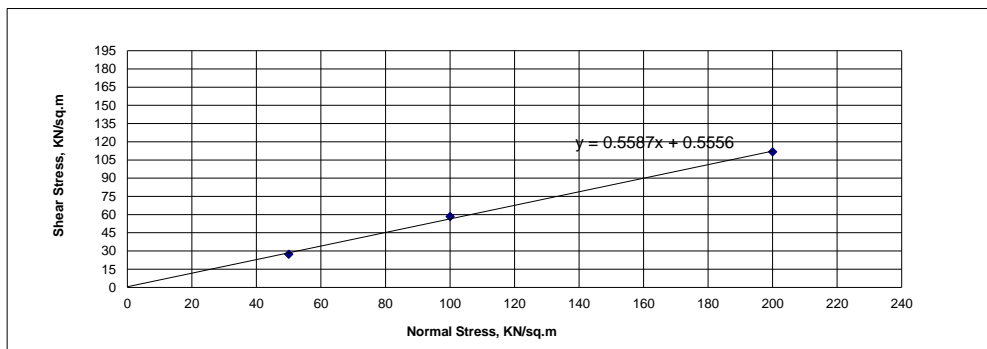
C                    4.72                    KN/m<sup>2</sup>  
 Φ                    29.85                    Degree

**Pulchowk Campus**  
Lalitpur.

**Direct Shear Test (IS : 2720 ( Part 13 ) - 1986)**

**Detailed Soil Investigation of Tuin khola.**

Bore Hole No:	1					
Sample No.:					PRG Factor, kN/Div.:	0.002
Depth (m)	S-4				Area m <sup>2</sup> :	0.0036
Test No. 1	1		2		3	
Wt. of Mould + sample (gm)	267.24					
Wt. of mould (gm)	107.09					
Density (gm/cm <sup>3</sup> )	1.78					
Weight of cont. + wet soil (gm)	119.65					
Weight of cont. + dry soil (gm)	110.38					
Wt of cont.(gm)	53.22					
Wt of dry soil(gm)	57.16					
Wt of water (gm)	9.27					
M/C, % :	16.22					
Horiz. Dial Reading 0.01mm) (x	Normal Load 50 kN/m <sup>2</sup>		Normal Load 100.0 kN/m <sup>2</sup>		Normal Load 200.0 kN/m <sup>2</sup>	
	Load Ring Dial Reading	Shear Stress KN/m <sup>2</sup>	Load Ring Dial Reading	Shear Stress KN/m <sup>2</sup>	Load Ring Dial Reading	Shear Stress KN/m <sup>2</sup>
0.00	0.00	0.00	0.00	0.00	0.00	0.00
25.00	5.00	2.78	9.00	5.00	11.00	6.11
50.00	14.00	7.78	15.00	8.33	18.00	10.00
100.00	19.00	10.56	24.00	13.33	28.00	15.56
125.00	26.00	14.44	35.00	19.44	39.00	21.67
150.00	35.00	19.44	44.00	24.44	52.00	28.89
175.00	37.00	20.56	51.00	28.33	65.00	36.11
200.00	37.00	20.56	58.00	32.22	82.00	45.56
250.00	43.00	23.89	62.00	34.44	97.00	53.89
300.00	44.00	24.44	69.00	38.33	114.00	63.33
350.00	49.00	27.22	73.00	40.56	131.00	72.78
400.00	48.00	26.67	78.00	43.33	139.00	77.22
450.00			81.00	45.00	148.00	82.22
500.00			88.00	48.89	160.00	88.89
550.00			95.00	52.78	171.00	95.00
600.00			99.00	55.00	186.00	103.33
700.00			105.00	58.33	201.00	111.67
800.00			103.00	57.22		
900.00						
1000.00						
1100.00						
1200.00						



C            0.56      KN/m<sup>2</sup>  
Φ            29.19      Degree

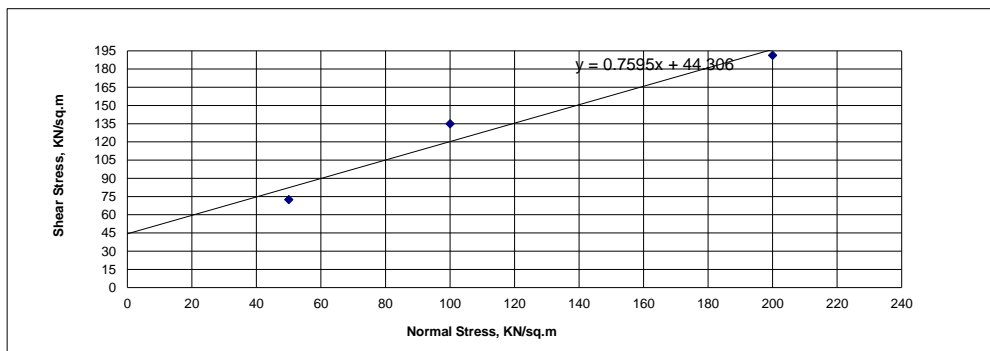
**Pulchowk Campus**

Lalitpur.

**Direct Shear Test (IS : 2720 ( Part 13 ) - 1986)**

**Detailed Soil Investigation of Tuin khola.**

Bore Hole No:	2					
Sample No.:					PRG Factor, kN/Div.:	0.00725
Depth (m)	S-4				Area m <sup>2</sup> :	0.0036
Test No. 1	1		2		3	
Wt. of Mould + sample (gm)	266.96					
Wt. of mould (gm)	107.09					
Density (gm/cm <sup>3</sup> )	1.78					
Weight of cont. + wet soil (gm)	119.00					
Weight of cont. + dry soil (gm)	116.23					
Wt of cont.(gm)	30.65					
Wt of dry soil(gm)	85.58					
Wt of water (gm)	2.77					
M/C, % :	3.24					
Horiz. Dial Reading 0.01mm) (x	Normal Load 50 kN/m <sup>2</sup>		Normal Load 100.0 kN/m <sup>2</sup>		Normal Load 200.0 kN/m <sup>2</sup>	
	Load Ring Dial Reading	Shear Stress KN/m <sup>2</sup>	Load Ring Dial Reading	Shear Stress KN/m <sup>2</sup>	Load Ring Dial Reading	Shear Stress KN/m <sup>2</sup>
0.00	0.00	0.00	0.00	0.00	0.00	0.00
25.00	3.00	6.04	7.00	14.10	14.00	28.19
50.00	8.00	16.11	12.00	24.17	19.00	38.26
100.00	10.00	20.14	17.00	34.24	25.00	50.35
125.00	12.00	24.17	19.00	38.26	31.00	62.43
150.00	14.00	28.19	25.00	50.35	38.00	76.53
175.00	16.00	32.22	31.00	62.43	48.00	96.67
200.00	18.00	36.25	35.00	70.49	55.00	110.76
250.00	20.00	40.28	39.00	78.54	67.00	134.93
300.00	24.00	48.33	46.00	92.64	74.00	149.03
350.00	29.00	58.40	51.00	102.71	85.00	171.18
400.00	34.00	68.47	58.00	116.81	88.00	177.22
450.00	36.00	72.50	67.00	134.93	95.00	191.32
500.00	35.00	70.49	65.00	130.90	94.00	189.31
550.00					93.00	187.29
600.00						
700.00						
800.00						
900.00						
1000.00						
1100.00						
1200.00						



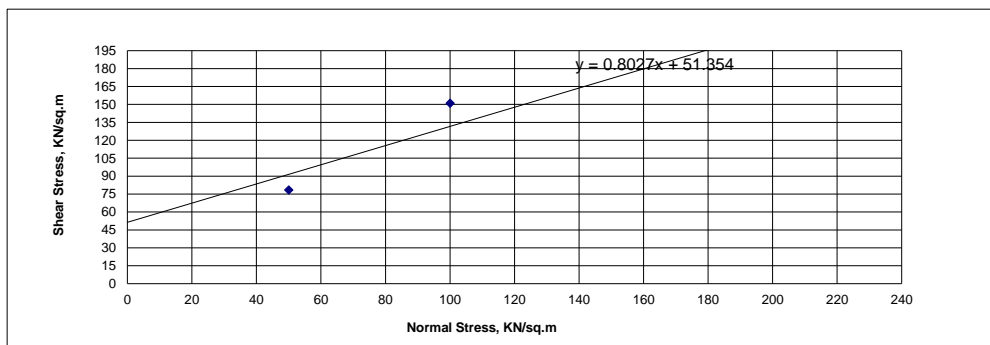
C            44.31      KN/m<sup>2</sup>  
 Φ            37.22      Degree

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**Direct Shear Test (IS : 2720 ( Part 13 ) - 1986)  
Detailed Soil Investigation of Tuin khola.**

Bore Hole No:	2					
Sample No.:					PRG Factor, kN/Div.:	0.00725
Depth (m)	S-5				Area m <sup>2</sup> :	0.0036
Test No. 1	1		2		3	
Wt. of Mould + sample (gm)	267.62					
Wt. of mould (gm)	107.09					
Density (gm/cm <sup>3</sup> )	1.78					
Weight of cont. + wet soil (gm)	132.12					
Weight of cont. + dry soil (gm)	127.32					
Wt of cont.(gm)	32.21					
Wt of dry soil(gm)	95.11					
Wt of water (gm)	4.80					
M/C. % :	5.05					
Horiz. Dial Reading (x 0.01mm)	Normal Load 50 kN/m <sup>2</sup>		Normal Load 100.0 kN/m <sup>2</sup>		Normal Load 200.0 kN/m <sup>2</sup>	
	Load Ring Dial Reading	Shear Stress KN/m <sup>2</sup>	Load Ring Dial Reading	Shear Stress KN/m <sup>2</sup>	Load Ring Dial Reading	Shear Stress KN/m <sup>2</sup>
0.00	0.00	0.00	0.00	0.00	0.00	0.00
25.00	5.00	10.07	7.00	14.10	9.00	18.13
50.00	9.00	18.13	14.00	28.19	28.00	56.39
100.00	16.00	32.22	23.00	46.32	35.00	70.49
125.00	17.00	34.24	30.00	60.42	42.00	84.58
150.00	24.00	48.33	38.00	76.53	50.00	100.69
175.00	28.00	56.39	47.00	94.65	59.00	118.82
200.00	37.00	74.51	55.00	110.76	67.00	134.93
250.00	39.00	78.54	62.00	124.86	76.00	153.06
300.00	38.00	76.53	69.00	138.96	93.00	187.29
350.00	37.00	74.51	75.00	151.04	102.00	205.42
400.00						
450.00						
500.00						
550.00						
600.00						
700.00						
800.00						
900.00						
1000.00						
1100.00						
1200.00						



C            51.35    KN/m<sup>2</sup>  
 Φ            38.75    Degree

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**UNIAXIAL UNCONFINED COMPRESSION STRENGTH OF ROCK SAMPLE**

As Per IS Code-9147:1979

**Detailed Soil Investigation of Tuin khola.**

S.No	Sample Hole No.	length	breadth	height	Area	Crushing Load	Ratio H/D	Correction factor (CF)	Compressive Strength of Core	Corrected Compressive Strength
		mm	mm	mm	mm <sup>2</sup>	KN	N	CF	N/mm <sup>2</sup>	N/mm <sup>2</sup>
1	SH -1	54.1	54.1	54.1	2926.81	235.90	1.00	1.00	80.60	80.60
2		54.3	54.3	54.3	2948.49	194.70	1.00	1.00	66.03	66.03
3		54.1	54.1	54.1	2926.81	196.30	1.00	1.00	67.07	67.07
4		54.2	54.2	54.2	2937.64	185.80	1.00	1.00	63.25	63.25
5		54.3	54.3	54.3	2948.49	204.60	1.00	1.00	69.39	69.39
4	SH -2	55.6	55.6	55.6	3091.36	91.50	1.00	1.00	29.60	29.60
5		55.5	55.5	55.5	3080.25	107.00	1.00	1.00	34.74	34.74
6		51	51	51	2601.00	83.00	1.00	1.00	31.91	31.91

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**Point Load Test Results on Core Samples**

As Per IS Code-8764:1998

**Detailed Soil Investigation of Tuin khola.**

S.No.	Drill Hole No.	SAMPLE	Test	Rock Size		Load at Failure P	Equivalent Dia of core		Uncorrected Point Load strength Is	F	Point Load strength Index Is (50)
				Width	Height		De <sup>2</sup>	De			
		m		mm	mm	Kpa	mm	mm	Mpa		Mpa
1	SH -1	A-1	Diametrical	54.35	40.48	7726.96	3,570.27	59.75	2.46	1.083	2.66
			Axial	40.48	80.10	8789.66	4,128.40	40.48	2.42	0.909	2.20
		A-2	Diametrical	54.20	47.55	7566.32	4,182.25	64.67	2.05	1.123	2.31
			Axial	54.55	61.88	8443.06	4,297.88	54.55	2.23	1.040	2.32
2	SH -2	B-1	Diametrical	55.05	52.61	4488.67	4,699.88	68.56	1.08	1.153	1.25
			Axial	54.61	63.88	4299.86	4,441.67	54.61	1.10	1.040	1.14
		B-2	Diametrical	54.90	50.40	3975.48	4,490.18	67.01	1.00	1.141	1.15
			Axial	54.40	77.10	3814.76	5,340.26	54.40	0.81	1.039	0.84
		B-3	Diametrical	54.60	51.41	3206.29	4,555.13	67.49	0.80	1.145	0.91
			Axial	54.41	78.00	4241.30	5,403.59	54.41	0.89	1.039	0.93

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**Unit Weight of soil**

Detailed Soil Investigation of Tuin khola.

	Sample No.	Wt. of mould (kg)	Wt. of mould + sample(kg)	Thickness (cm)	Length of sample (cm)	weight of sample(KN)	Volume(m3)	Unit Weight(KN/m3)
1	1.0	135.670	295.130	2.500	6.000	1.564	0.00009	17.375
	2.0	135.670	309.080	2.500	6.000	1.701	0.00009	18.895
	3.0	135.670	307.020	2.500	6.000	1.680	0.00009	18.671
	4.0	135.670	311.150	2.500	6.000	1.721	0.00009	19.121
2	1.0	135.670	351.020	2.500	6.000	2.112	0.00009	23.465
	2.0	135.670	358.150	2.500	6.000	2.182	0.00009	24.242





## **ANNEX B: Photographs**

Site Photo



site location - photo1



<p>Tuin khola area <b>After</b> massive landslide, pic. Date:2082-06-21</p>	<p>Tuin khola Area (CH 33+900) <b>Before</b> massive landslide, pic. Date:2082-03-08</p>
	
<p>Failure slope, @ center portion</p>	<p>Photos during sample collection</p>
	
<p>Slope @ ch-00+060 Exposed weather phyllite and quartzite</p>	<p>Confirming Dimensions, and Access Points Works under Inspection</p>

	
<p>Slope @ chainage- 00+020</p>	<p>Slope @ chainage- 00+040</p>
	
<p>Slope @ chainage- 00+100</p>	<p>Slope @ chainage- 00+120</p>

## **ANNEX C: Publication**



# 17th IOE Graduate Conference

## Certificate Of Participation



This certificate is awarded to

**Aayush Yadav**

in recognition of an invaluable contribution as

**Paper Presenter**

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Conference Convener

Prof. Dr. Sushil Bahadur Bajracharya  
Dean, Institute of Engineering

# Stability Assessment and Mitigation of Cut Slope Landslides: A Case Study of Twin Khola, Narayanghat-Mugling Road Section

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## Abstract

The Failures in the roadside at cut slopes in the Narayanghat-Mugling (NM) Road happen all the time. This study investigates a recent landslip due to soil erosion at Twin Khola along the NM Road section. Geological mapping and rock mass rating (RMR) were used to describe the properties of the rock formation at the places before it breaks down. We did field work by horizontal drilling boreholes in one place on the slope to get samples of soil and rock. Analyse the different crucial factor of site and perform the laboratory testing collected sample. Characterise the geological data of different place and the data are used for kinematics analysis of these slopes. Geophysical design parameter are to be considered for analysis. GeoStudio SLOPE/W is used to model different scenarios in numerical modelling with limit equilibrium methods. Comparison of factor of safety with the different condition of slope instability. This study investigate the foliation plane, shear zone, and rock characteristic, slope instability.

Based on the collected field and geotechnical characteristics, the current stability condition of the slope is evaluated to support informed mitigation and management measures in hilly regions.

## Keywords

Geostudio; Instability; Landslip; Model; Slopes

## 1. Introduction

Landslides represent a significant geohazard in mountainous regions, frequently disrupting infrastructure, causing economic losses, and posing risks to human. This highway is of considerable strategic importance, serving as the sole transportation route in the Lesser Himalayan hilly terrain for public transit, while also connecting numerous religious sites [1]. Large and complex landslides are related to deep rock weathering followed by the intervention of geological structures such as joints, faults, and fractures as shown in figure 1. The case of Narayanghat Mugling (NM), Dharan-Dhankuta, Banepa Bardiwas, Lamosangu-Jiri, Krishnabhir road sections are some examples of slope instability problems. In addition, presently many road slope instabilities are also observed and heard, which shows the need for awareness of geotechnical and environmental aspects in management. Landslides represent a significant geohazard in mountainous regions, frequently disrupting infrastructure, causing economic losses, and posing risks to human. The Twin Khola bridge area is known for its susceptibility to frequent rainfall-induced landslides, a result of the fragile geology of the Himalayas, which has been worsened by infrastructure development such as road construction. Local terrain comprises highly-jointed, fractured, and weak metamorphic rocks such as schist, phyllite, and quartzite. The geological formation of twin khola area is Sangram Formation (sg) in which thinly foliated, phyllite occasionally interfoliated with thin to thick bands of quartzite and limestone. This sg falls in Main Boundary Thrust (MBT) system that separates the Siwaliks (Sub-Himalayan Zone) to the south from the Lesser Himalayan Zone to the north. During heavy monsoon rainfall, these formations get saturated, which increases pore-water pressure in them and drastically reduces their stability. Construction efforts for the

NM road involved cutting into such steep and unstable hillsides; improper slope cutting and poor drainage practices have further significantly amplified this natural instability, turning it into one of the highway's most hazardous zones. The current 60 m bridge opened traffic in late 2024 or early 2025 and replaced an earlier one in a spot where widening of the road has created a new, expansive zone susceptible to landslide failure, now originating higher up the slope.

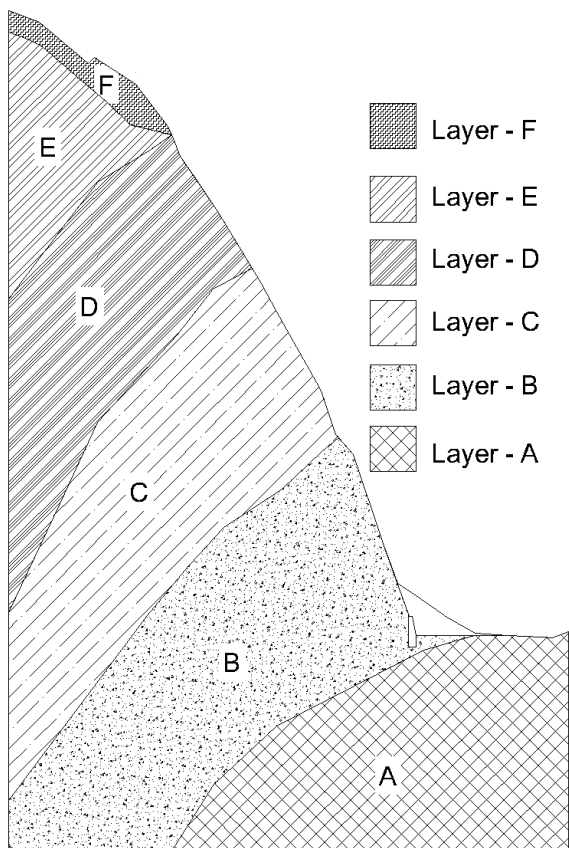
### 1.1 Statement of the Problem

A recent landslide has occurred along twin khola, NM Road, indicating potential slope instability arising from both anthropogenic and natural causes. Preliminary observations suggest that toe-cutting for road expansion, unfavorable geological conditions, and light rainfall may have jointly contributed to the slope failure. Infrastructure development, specifically the construction and widening of the NM road, has severely destabilized naturally fragile, weak, and highly fractured Himalayan hillsides. The steep slope cutting and inadequate drainage in this geologically unstable area interact with heavy monsoon rainfall, leading to saturated soil and rock, increased pore-water pressure, decrease the shear strength and frequent, destructive landslides that compromise the safety and functionality of the crucial highway and its new bridge. Such geological conditions, characterized by fragile geology, make these areas highly susceptible to landslides and erosion, further exacerbated by poorly engineered road construction practices [2]. The Cross-section showing layer of foliation plane is shown in figure 2. Based on the foliation plane, here is a short description of each layer (A–F), technical terms. Layer A — Bedrock / Fresh Rock, represents the strongest and oldest rock mass at the base. Layer B — Moderately Weathered Rock, common location for shear zones or weak planes. Layer C — Moderately Weathered Rock,



**Figure 1:** Twin khola Area (CH 33+900) Before massive landslide

location for shear zones or weak planes, black deposition is seen outside. Layer D — Foliated Rock Zone, strong presence of foliation planes / planar discontinuities. Layer E — Weak, highly weathered Rock Mass, represents shear-affected and fractured rock fabric. Layer F — Loose surface soil / debris / colluvial soil/ boulder mix sand. The



**Figure 2:** Cross-section showing layer of foliation plane

motivation of this research is to execute the detail slope assessment work and clarify the cause of occurrence of the slide. Accordingly, to point out the appropriate mitigative

measures for slope stability.

### 1.2 Study Area

The proposed site is located at Twin khola, Mugling-Narayanghat road section. The co-ordinates of this location is 27.842978, 84.558697. The aerial view site is shown in figure 3.



**Figure 3:** Outline of overall area of study area

### 1.3 Scope of the work

- To collect existing data, maps and information relevant to landslide and slope failure of the study area.
- To visit site and carry out the study of topography, geology and geotechnical feature of the slope.
- To evaluate the existing slope's condition

## 2. Methodology

The flow digram (figure 4) illustrate the overall sequences of this study which consist of following approaches.

### 2.1 Field Investigation

**Geophysical study:** The Microtremor Array Measurement (MAM) is a non-invasive geophysical test used to determine subsurface shear wave velocity profiles by recording natural ground vibrations (ambient seismic noise). The results of geological observations indicates that the landslide is structurally controlled and influenced by weakened phyllite/quartzite and saturation effects during monsoon. phyllite, metasandstone and quartzite [3]. Toe erosion by Twin Khola accelerates instability. High weathering, presence of seepage, unfavorable discontinuity orientation and low shear

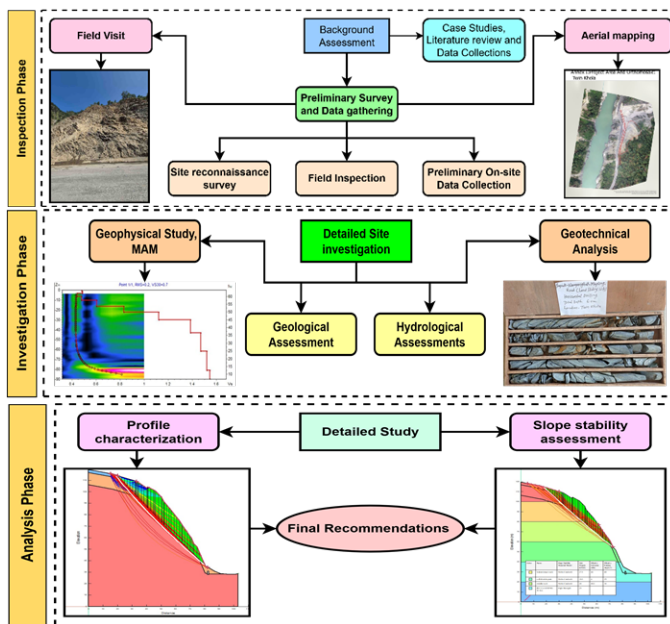


Figure 4: Research Methodology

wave velocity zones collectively contribute to sliding. Geophysical methods proved effective in mapping subsurface weakness without extensive drilling. **Geotechnical study:** The horizontal drilling is a specialized technique used for investigating, stabilizing, and monitoring rock slopes. Horizontal drilling is carried out laterally into the rock mass, typically perpendicular to the slope face, allowing us to evaluate and improve the internal mechanical condition of potentially unstable rock masses. Assess discontinuities, joints, fractures, and weathered zones inside rock masses **Hydrological study:** Rainfall serves as the primary determinant for assessing landslide hazard potential. Situated within the Lesser Himalaya, where rainfall is frequent, and the project site experiences significant rainfall variability typical of Nepal. **Rock Mass Characterization:** RMR will be comprehensively assessed at various representative points along the cut slopes parameters including Uniaxial Compressive Strength, Rock Quality Designation, joint spacing, persistence, separation, roughness of joint surfaces, infilling material characteristics and the degree of weathering Geological Report showing RMR of the Cut Slopes along the NM Road NH44 Highway at the Twin khola area, landslide.

## 2.2 Laboratory Test

For colluvial deposits containing sand, gravel, and boulders, the laboratory tests of the material to be performed. The characteristic of soil debris material is highly heterogeneous. The test of brittle type of rock is crucial for sampling. These are the following laboratory test to be done for this study.

- Particle Size Distribution
- Specific Gravity
- Natural Moisture Content
- Direct Shear Test
- Bulk Density/Unit Weight

- Modified / Standard Proctor Test
- Uniaxial Unconfined Compressive Strength
- Point Load Index Test

## 2.3 Stability Assessment through Numerical Modeling

The comprehensive geotechnical and geological data obtained from both field investigations and laboratory analyses will be critically utilized for numerical modeling. This approach aims to accurately assess the slope stability and precisely identify potential failure mechanisms. While direct analysis, the parameters derived from careful visual interpretation of site photos (e.g., slope angles, geological features, discontinuity orientations, presence of tension cracks) will be crucial for defining the model.

## 2.4 Road Construction and Anthropogenic Landslides

The study documented that landslide triggering by road construction in Nepal involves: (1) excessive cutting heights and steep slopes, (2) absence of slope protection measures, (3) lack of drainage systems, (4) side-casting of excavated spoil, and (5) vegetation removal. The Department of Roads guidelines recommend slope cuts of 1:0.3 for hard rock and 1:0.8 for soils.

## 3. Result and Discussion

### 3.1 Main Causes for occurrence of slide

The causes for occurring the observed several slides are light rainfall for long period, presence of the loose colluvial deposits on steep slope and poor surface and subsurface drainage. Highly weathered and fractured bedrock of phyllite with quartzite and limestone composite the landslide. It is basically due to mainly by soil erosion.

- Activation of the local fault, seen on the left side of the slide area and chances of occurring plane failure along foliation plane.
- Steep slope with loose colluvial deposits on the crown part of the slide.
- Dipping of the natural hill slope and foliation plane is not in same direction.

### 3.2 Interpretation of MAM

MAM was carried out at cut slope of road velocity map along this profile as shown in figure 5. The depth and characteristics of the soil strata obtained from the MAM test are depicted for the different layers and presented in Table 1. Empirical expression for the allowable soil pressure  $q_a$  underneath shallow foundations based on seismic velocities can be obtained by following the systematic boundary value approach used earlier by Keceli (2012). When the ultimate bearing capacity is expressed in terms of shear wave velocity, the ultimate bearing capacity may be written as

$$Q_{ult} = g \rho V_s T \quad (1)$$

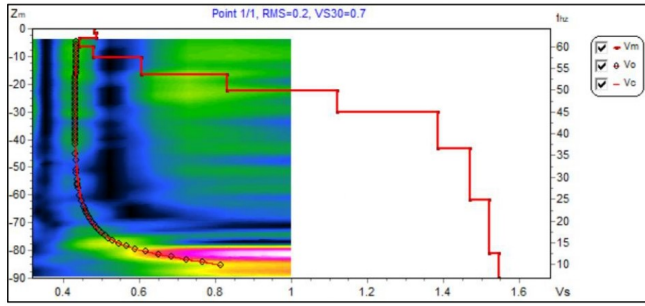


Figure 5: Dispersion curve of MAM

Since  $\gamma = g\rho$ , Eq. (1) becomes

$$Q_{ult} = \gamma V_s T \tag{2}$$

where  $g = 9.81 \text{ m/s}^2$  is the acceleration due to gravity,  $\rho$  is the density of the soil beneath the foundation,  $V_s$  is the shear wave velocity,  $\gamma$  is the unit weight of the soil, and  $T$  is a time parameter that is determined through a calibration process. For this purpose, a typical hard rock formation is assumed to exist beneath the foundation with parameters suggested by Keceli (2012). It is well known that the allowable bearing capacity  $q_a$  is defined as the ratio of the ultimate resistance of the earth structure to the FS:

$$q_a = \frac{Q_{ult}}{F_s} \tag{3}$$

After a series of calibration analyses, the safety factor is taken as  $F_s = 1.4$  for  $V_s \geq 4000 \text{ m/s}$  and  $F_s = 4.0$  for  $V_s \leq 750 \text{ m/s}$ . According to Keceli (2012), it is also well established that the presence of groundwater has a significant influence on the bearing capacity of soils[4].

Top layer with shear wave velocity of 443 m/s - 481 m/s up to the depth of 6.5m indicates colluvial deposit with gravels and boulders. The equivalent allowable bearing capacity of this layer is  $223 \text{ kN/m}^2$  -  $245 \text{ kN/m}^2$ .

Shear wave velocity ranging between 479 m/s - 605 m/s up to the depth of 16.46m indicates highly weathered and fractured bedrock of phyllite with quartzite and limestone. The equivalent allowable bearing capacity of this layer varies from  $239 \text{ kN/m}^2$  -  $302 \text{ kN/m}^2$ .

Shear wave velocity ranging between 605 m/s - 1122 m/s up to the depth 30.03m indicates fractured to competent bedrock of phyllite with quartzite and limestone. The equivalent allowable bearing capacity of this layer is  $302 \text{ kN/m}^2$  -  $589 \text{ kN/m}^2$ .

Shear wave velocity ranging between 1386 m/s - 1521 m/s below the depth of 30.03m indicates competent bedrock of phyllite with quartzite and limestone. The equivalent allowable bearing capacity of this layer is  $589 \text{ kN/m}^2$  -  $897 \text{ kN/m}^2$ .

### 3.3 Interpretation of Geotechnical data Findings of core drillings

Drill hole is done on the downhill slope of the road alignment. Up to 6.0 m depth, horizontal drilling have been perform.

Table 1: Interpretation MAM test data

Depth (m)	Type of Soil
6.50	Colluvial deposit with gravels and boulders
16.46	Highly weathered and fractured bedrock of phyllite with quartzite and limestone
30.03	Fractured to competent bedrock of phyllite with quartzite and limestone.
>30.03	Competent bedrock of phyllite with quartzite and limestone.

Recovered samples were preserved in core boxes for logging and laboratory analysis. In addition to the core samples, debris materials were collected from the crown area of the slope, representing colluvial deposits composed of a mixture of boulders and sand and test results are tabulated in table 2. Below the colluvial layer, the subsurface profile is dominated by quartzite and phyllite bedrock. The bedrocks are hard and fractured with maximum 25% core recovery is possible. The unconfined compressive strength (UCS), Point Load index (PI) and unit weight (Unit Wt.) of rock sample are outline in table 3. No Ground water table was observed during the exploration.

Table 2: Test Results of colluvial deposit

Gravel %	Sand %	Fines %	NMC %	Sp.gravity (G)	c kN/m <sup>2</sup>	phi Degree	Unit Wt. kN/m <sup>3</sup>
62.64	26.33	11.03	14.02	2.66	5.28	25.28	16.94
27.13	64.97	7.90	20.54	2.65	4.72	29.85	17.15
50.03	39.54	10.42	8.65	2.66	-	-	16.93
59.38	34.76	5.86	19.04	2.67	0.56	29.19	16.83

Table 3: Test Results of rore and rock block sample

Sample details	Type	UCS	PI	Unit Wt.
		MPa	MPa	KN/m <sup>3</sup>
Block sample 1	Phyllite	42.6	2.01	26.7
Block sample 2	Phyllite	36.5	1.93	26.7
Drill sample, 1.0m	Phyllite	x	2.19	x
Drill sample, 2.0m	Phyllite	45.9	x	26.2
Drill sample, 4.0m	Phyllite	38.6	x	26
Drill sample, 5.0m	Phyllite	x	1.66	x
Block sample 1	Quartzite	78.1	4.37	26.3
Block sample 2	Quartzite	92.5	5.06	26.3
Block sample 3	Quartzite	82.3	4.64	27.5
Block sample 4	Quartzite	69.27	2.37	27.82
Block sample 5	Phyllite	32.08	1.04	26.64

- The debris material is predominantly a coarse-grained soil mixture (silty gravels and sands). From the laboratory test, it is found that cohesion value is (4-5) and friction angle of (25-29.5) degree. The low cohesion values, especially in the presence of water (high moisture content), mean these soils could be highly susceptible to shear failure, a key characteristic of landslide-prone debris.
- A comprehensive analysis of the mechanical properties of rock samples, including UCS, PI, and Unit Weight.

- UCS decreased with depth in the available measurements, from 45.9 MPa at 2.0 m to 38.6 MPa at 4.0 m — an approximate 16% drop over that interval.
- Quartzite has a much higher average UCS (80.54 MPa) than Phyllite (37.06 MPa), making Quartzite roughly 2.2 times stronger on average.
- Quartzite also shows a higher PI Value (4.11) compared with Phyllite (1.66), suggesting different deformation behaviors between the rock types.
- Average unit weight is nearly identical for Quartzite and Phyllite (about  $26.9 \text{ kN/m}^3$ ), indicating the strength difference between them is not due to bulk density.

### 3.4 Slope Stability Analysis

The five representative slope sections along the Twin Khola corridor were subjected to numerical modeling in order to assess their stability under the existing site conditions. The surface with the lowest FS for each slope was determined to be the critical slip surface. The critical slip surface being the specific failure plane within a slope that yields the minimum FS as sketch in figure 6. The hill side is particularly unstable because of a thick colluvium deposit followed by highly weathered rock mass that is prone to shifting of whole mass of slide [5]. The slope stability analysis has been done based on

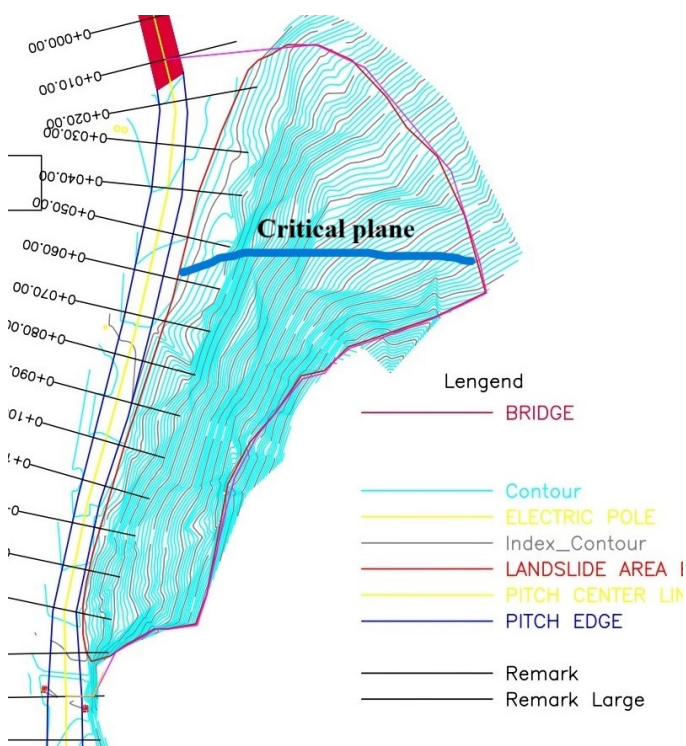


Figure 6: Critical Path for the Stability Analysis

the relation between the dipping of the natural hill slope and wedges formed by intersection of the foliation plane and joints. Slope cut into a homogeneous and isotropic rock mass, which increases cohesion of the rock that could raise the slope angle from 40 to 50 [6]. Design considerations for slope stability analysis include material properties like shear strength, unit weight, and pore water pressure, as well as external factors such as seismic loads, slip surface and

external cracks. In SLOPE/W, numerical modeling involves defining geometry, materials using various constitutive models and boundary conditions, followed by simulating a range of scenarios, including reinforcement techniques, to determine the critical slip surface and Factor of Safety (FS). The table 4 illustrate very steep slope and unfavorably

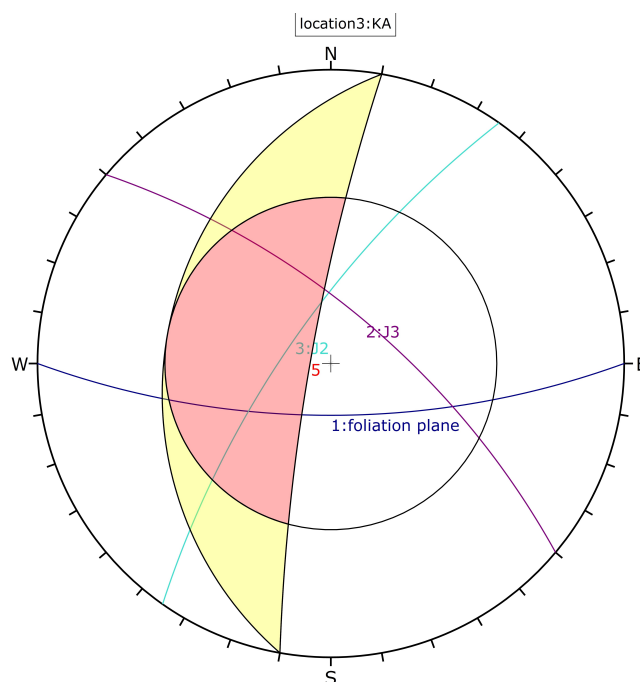


Figure 7: Kinematic Analysis of failure slope

oriented foliation and joints indicate a high risk of planar or wedge failure, which should be confirmed through stereonet kinematic analysis as shown in figure 7. In wedge failure, two intersecting planes are present which dip towards each other. Movement takes place by sliding along both planes in the direction of the line of intersection [7]. The 66.67% of the total area is critical for failure, specifically wedge sliding. Remaining wedges seem to be stable and very less chances to occur the failure. The instabilities are structurally controlled and less potential to failure. The analytical process involves

Table 4: Kinematic Analysis Parameter for stereographic Projection

S.No.	Dip	Dip Direction	Label
1	70	180	Foliation Plane
2	73	305	J2
3	69	40	J3
4	82	280	Hill Slope

identifying this surface through investigation and modeling in slope/w as shown in figure 8, then designing appropriate mitigation to ensure stability. The table 5 shows that the original slope during rainfall is unsafe because its FS is less than 1 under both normal and seismic conditions, leading to an unstable slope.

- Slope protection with RCC wall rock Bolt and high tensile wire mesh during the monsoon, FS is improved up to 84.21%.

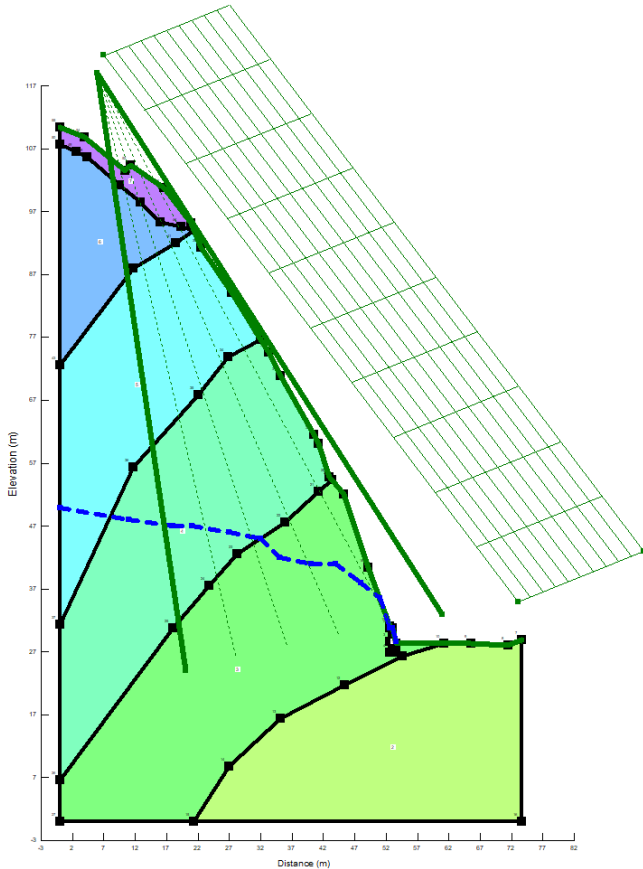


Figure 8: Interpreted Geostudio model of the slope section

Table 5: Factor of safety including Slope Condition

S.No.	Slope Conditions	Factors of Safety		Remarks
		In-situ stress Conditions	Considering Seismic effect	
1	Slope after failure during rainfall	0.71	0.57	Unstable slope
2	Slope protection with RCC wall rock Bolt	1.23	1.05	Stable slope
3	Slope protection with RCC wall rock Bolt, with drainage	1.71	1.59	Stable slope
4	Slope protection with RCC wall rock Bolt, dry condition	2.09	1.92	Stable slope

- Slope protection with RCC wall rock Bolt and high tensile wire mesh with drainage, FS is improved up to 178.95%.
- Slope protection with RCC wall rock Bolt and high tensile wire mesh, at dry condition, FS is improved up to 236.84%.
- Slope protection with RCC wall rock Bolt and high tensile wire mesh, comparison with second condition without drainage path, FS is improved up to 51.43%.

In general the FS are vigorously decreasing during the monsoon season. The FS equation is a variation of the Mohr-Coulomb equation, where the soil's shear strength parameters are reduced by the FS value until the slope is at the point of failure (limit equilibrium state).

$$F_s = \frac{\text{Available Shear Strength}}{\text{Mobilized Shear Stress}} = \frac{c' + (\sigma_n - u_w) \tan(\phi')}{\tau} \quad (4)$$

The analysis seeks to find the minimum FS for the most critical slip surface. The Morgenstern-Price method satisfy all three conditions of equilibrium simultaneously, making them generally more accurate. These methods introduce a relationship between the interslice normal force ( $E$ ) and interslice shear force ( $X$ ):

$$X = \lambda \cdot f(x) \cdot E \quad (5)$$

where:

$f(x)$  = assumed interslice force function,

$\lambda$  = scaling factor),

In its central zone, where thick beds of stronger rocks like amphibolite and quartzite are exposed, the dominant failure is by block toppling and the upper slope are thinly foliated phyllite and slate exert flexural and multiple rock topples. The instability is more closely related to regional tectonic control by nearby thrust faults, with subsequent gravitational deformation, unfavorably oriented foliation and joint sets, intensely sheared and fractured zones, small faults within the slide mass and toe undercutting; these all together govern the initiation and continued evolution of the massive landslide.

#### 4. Conclusion

This study demonstrates that foliation planes, shear zones, and low-quality rock mass conditions control the Twin Khola roadside slope's structural stability and geotechnical weakness, causing frequent instability along the Narayanghat–Mugling Road. Quartzite and phyllite of the Tistung Formation. SLOPE/W modeling confirms reduced stability under wet and groundwater-influenced conditions, while laboratory, geophysical, and kinematic analyses show low material strength and unfavorable discontinuity orientations. The primary failure surface is located close to the colluvium–bedrock interface, highlighting the necessity of slope monitoring and focused engineering stabilization to improve road safety.

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