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Cut Slope Stability Analysis on Terrain with Slope at Besishahar, Lamjung

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Abstract

Building construction on sloped terrain presents a myriad of challenges that necessitate specialized engineering solutions and careful planning. Though the cut slopes are prone to failure, the increase in urbanization makes construction in such landscape inevitable. The complex topography of this region poses significant scientific and technological challenges, demanding innovative solutions for slope stability analysis and effective mitigation measures. This study incorporates geotechnical and geological study of the Manab Sewa Ashram Site in Besishahar, the cut slope failure analysis using Limit Equilibrium Method and Kinematic failure analysis and suggesting mitigating measure. Three different profiles with critical slopes were selected to represent the overall area for Slope Stability modeling and Kinematic Failure Analysis. The calculation of Factor of Safety for slope stability was done using Slide v.6.020 software under Rocscience package. Since non-engineered cutting of slopes is predominantly existing in the context of Nepal, slope management contributes by majorly focusing on the safety of the structure on or around the slopes.

Keywords: Slope stability, Slide software analysis, Limit equilibrium analysis, Slope analysis in Besishahar Lamjung, Cut Slope, Mitigation for Cut Slope.

1 Introduction

The diverse topography of Nepal presents urbanization challenges due to a shortage of suitable land for construction. The country encompasses hilly, flat, and mountainous regions, creating a demand for construction sites exacerbated by urban development. Sloped terrain in Nepal, susceptible to erosion and landslides, requires crucial attention to stability during construction. Soil stability is paramount for the structural integrity of engineering projects, with unstable soil posing risks of settlement or structural failure. Addressing slope stability is vital for the success and resilience of construction activities in the varied landscape of Nepal.

1.1 Study Area

The selected research site is about 3 km northwest from Besishahar, Lamjung, Nepal (28°13'46.7"N 84°22'14.3"E) located besides the road joining Upallo Samdi Village to Besishahar where the structure under construction was Manab Sewa Ashram buildings. The coverage area of the site is about $100 * 100$ sq.m.

1.2 Review on geology of the study area

The research area lies in Lesser Himalaya which consists of these four formations:

Seti Formation: The rock of the Seti Formations comprises the grey, greenish grey, gritty Phyllite grill stones with conglomerate and white massive quartzite. In the upper part of the formations, the basic rock intrusion is found. The project area lies within this formation.

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Naudanda Formation: The rock of the Naudanda Formations comprises the white massive, fine to medium grained quartzite with ripple marks interbedded with green phyllites. In upper parts basic intrusion is noted.

Ghan Phokhara formations: The rock of this formations comprises the black to grey carbonaceous slate and green slate.

Chour Carbonates: The Carbonate succession comprises the white to grey, coarse, chemically weathered carbonate beds with grey compact dolomite and dolomitic limestone interbedded with shale beds (Pokhrel & Pathak, 2018).

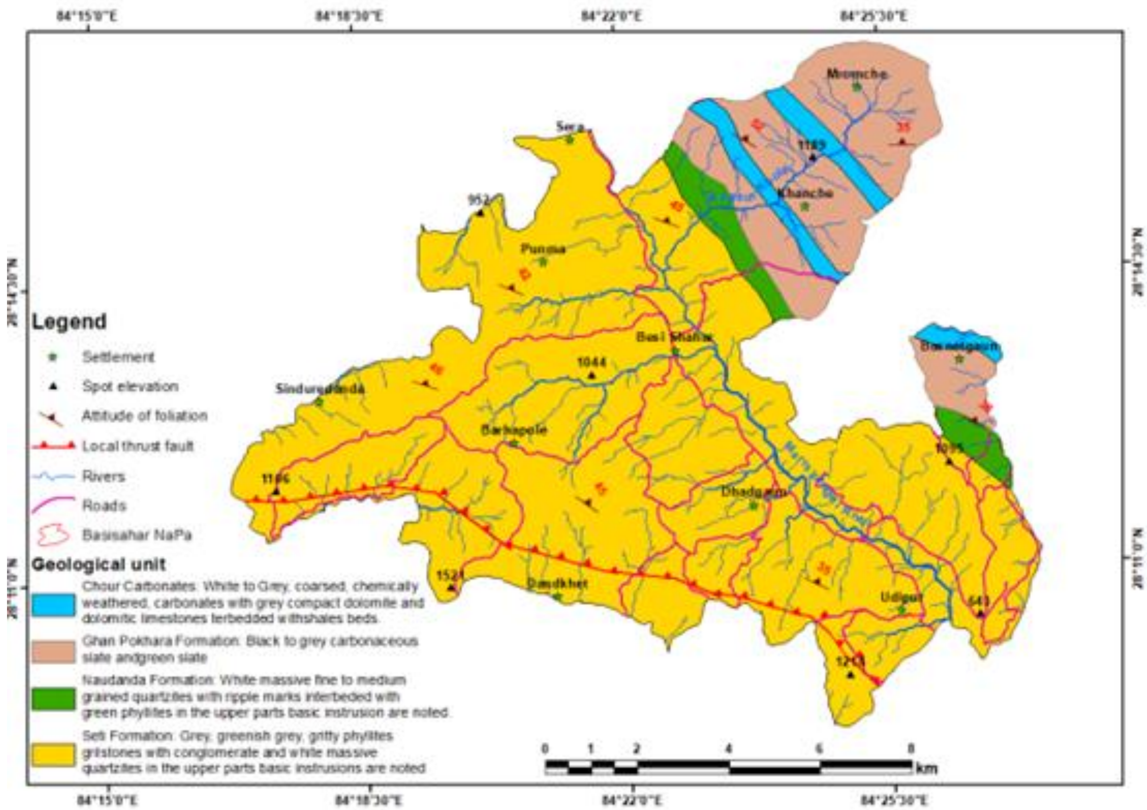

Figure 1. Geological map of Study area (Source: GSI, 2023)

1.3 Slope stability

The stability of a slope, whether natural or artificial, depends on various factors such as the geological and geotechnical characteristics of the slope materials; the slope angle, the presence of water, and the external loads acting on the slope. Engineering measures such as slope reinforcement techniques, drainage systems and careful consideration of material properties can help mitigate the risks associated with artificial slope instability. Various engineering techniques are employed, such as slope reinforcement, retaining walls, and drainage systems (Cruden et al., 1996).

The major factors that impact the stability of any slope are:

- Slope Geometry
- Subsoil Conditions

Slope failure occurs due to sudden or gradual loss of strength by the soil or to any disturbances created to the geometric conditions of the soil slope.

The stability analysis can be done either by published chart solution or computer analysis where the latter is preferred to conclude with a mitigation measure. Primarily for 2−D Analysis, the Limit Equilibrium (LEM) approach is used. Finite Element Method (FEM) and Boundary Element Method (BEM) are other methods which provide refined 2−D and 3−D analyses but require a relatively complete model of subsoils and parameters. The analysis in the report is based on LEM because of the simplicity of the method which is suitable for our slope failure site in Lamjung.

An investigation conducted by Zhang revealed that from among the 334 landslides that occurred in Wenchuan (China) during the 2008 earthquake, 35% were rock-soil slopes, suggesting that the rock-soil slope is ubiquitous and that particular attention should be paid to the potential hazards and risks of this type of slope (Liu et al., 2018). On initial assessment, the project site revealed to have a combination of both soil and rock elements in the slope. Various tests were carried out to determine the type and strength of both the soil and the rock.

1.4 Limit Equilibrium Method (LEM)

Limit equilibrium methods (LEMs) are utilized to analyze the stability of soil masses prone to gravitational sliding. These methods assess the necessary shear strength along a potential failure surface to maintain stability and compare it with the available shear strength. They examine transitional or rotational movements along assumed or identified potential slip surfaces beneath the soil or rock mass, particularly crucial in rock slope engineering for addressing block failures along distinct discontinuities. These methods operate by comparing resisting forces, moments, or stresses with destabilizing forces, resulting in a Factor of Safety (FOS) representing the ratio of shear strength or equivalent resistance to the shear stress required for equilibrium. If the FOS falls below 1.0, the slope is deemed unstable (Turner & Schuster, 1996).

1.5 Factor of safety

The factor of safety is defined as the ratio of the shear strength divided by the shear stress required for equilibrium of the slope:

$$F = \frac{shear\ strength}{shear\ stress\ required\ for\ equilibrium} \qquad \text{(Equation 1)}$$

Where, F= Factor of Safety (Duncan, 1996)

Safety factors that are less than 1 indicate of failure or, at minimum, the possibility of failure. Conversely, stability is represented by safety factors that are greater than 1. The selection of the appropriate safety factor for a particular slope depends on several factors, such as the quality of the data utilized in the analysis, which is contingent on the quality of subsurface investigations, laboratory and field testing, interpretation of field and laboratory data, construction control quality, and in some cases, the degree of completeness of information about the design problem.

1.6 Geological Strength Index (GSI)

The Geological Strength Index (GSI) is an outstanding rock mass classification system that employs various approaches to evaluate rock masses that are very weak and have numerous joints. This system is linked to the geo-mechanical properties of rock masses, including Hoek and Brown constants, deformation modulus, strength properties, and Poisson's ratio, to enable the appropriate design of engineering structures such as tunnels and caverns. The GSI values of rock masses are calculated using the empirical formula (Somodi et al., 2021).

2 METHODOLOGY

2.1 Topographic survey

Contour map of the project area was developed by Electronic Distance Measurement (EDM) method using total station and detailed topographic survey was carried out. Control points were set and temporary benchmarks were used. Coordinates of topographic features were recorded and cross sections of the slope area was developed.

2.2 Stereographic plot

A stereographic plot is a graphical representation used in structural geology to analyze and visualize three-dimensional orientations of geological features on a two-dimensional plane. It is particularly useful for studying the distribution of planar and linear features like bedding planes, faults, folds, etc. Stereographic projections help geologists understand the spatial relationships and orientations of geological structures.

2.3 Laboratory tests

Disturbed soil sampling and hand size rock sampling were done with the help of hammer and chisel and were air tightly packed in plastic sample bags from different locations with GPS location. For bulk density test, undisturbed samples were collected using core cutter method with small cylinders.

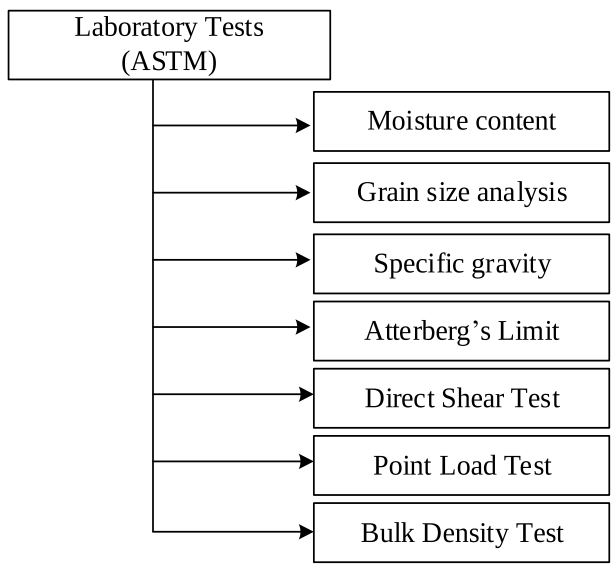

Figure 2. Laboratory Tests

The direct shear test is a fundamental laboratory experiment used in geotechnical engineering to assess the shear strength properties of soils. This information is essential for various geotechnical applications, including slope stability analysis, foundation design, and construction planning (Das, 2021). The ability of soil to resist shear forces influences the design of foundations, ensuring they can safely support the applied loads (Terzaghi, 1943). Cohesion and the angle of internal friction obtained from the direct shear test are key factors used in the Slide v.6.020 software in assessing stability of the slope.

The Point Load Test is particularly useful when dealing with large rock cores or when conventional uniaxial compression testing equipment is unavailable or impractical. However, it should be noted that the test provides an estimate of the uniaxial compressive strength and is not a direct substitute for more comprehensive testing methods like the Uniaxial Compressive Strength (UCS) test. The UCS test is generally more accurate but requires more time, effort, and equipment.

2.4 Slide software analysis

Slide v.6.020 software, by Rocscience is used to evaluate the stability of the slope. The software employs the Limit Equilibrium method, a commonly used approach in geotechnical engineering for assessing the factor of safety and determining whether a slope is stable or prone to failure.

In Limit Equilibrium techniques, slope stability is analyzed by first computing the factor of safety. This value must be determined for the surface that is most likely to fail by sliding, the so-called critical slip surface. Iterative procedures are used, each involving the selection of a potential sliding mass, subdivision of this mass into a series of slices, and consideration of the equilibrium of each of these slices by one of several possible computational methods. These methods have varying degrees of computational accuracy depending on the suitability of the underlying simplifying assumptions for the analyzed situation (Duncan, 1996).

The slope profile was plotted on the Slide software to proceed the modeling. The interface of soil and rock was also drawn as per slip surface observed in the field and ERT data was later incorporated. The required data was collected through the lab tests of materials. Slide software allows limit equilibrium analysis of soil slope. Bishop (1955) and Janbu's method was used to calculate the factor of safety for satisfying both the force and moment equilibrium. The material properties were defined as per the lab test data and Mohr-Coulomb material model was used for top soil along with generalized Hoek-Brown method for fractured rock and bed rock.

3 RESULTS AND DISCUSSIONS

The thorough site investigation, soil sampling, contour mapping, and software analysis have provided a complete understanding of the characteristics of the site. This comprehensive approach has given valuable insights into soil composition, site topography, and geotechnical conditions, enabling informed decision-making and strategic planning.

3.1 Contour map of the study area

The contour map of the site was developed out of which three lines were selected for the cross section to analyze the slope using slide software as shown in Figure 3.

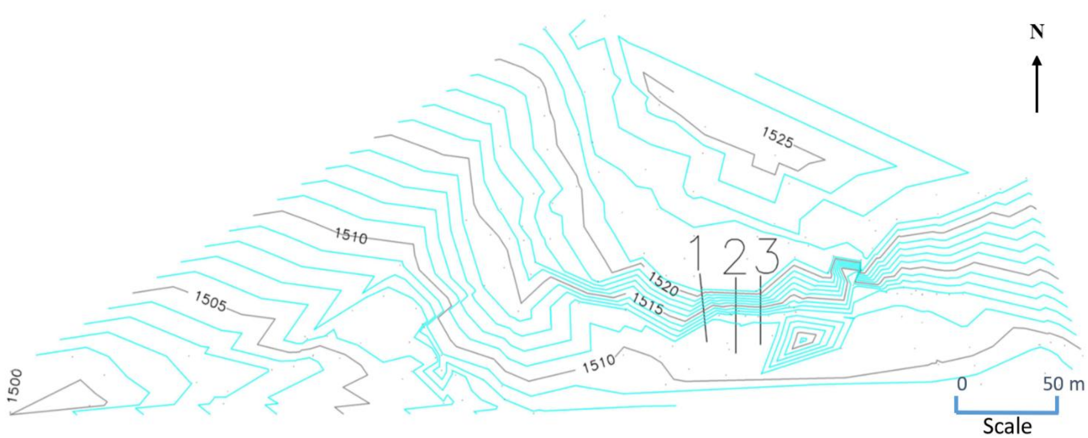

Figure 3. Contour map of study area showing planes of cross section

The detailed view of the cross section of the plane 2 is as shown in Figure 4.

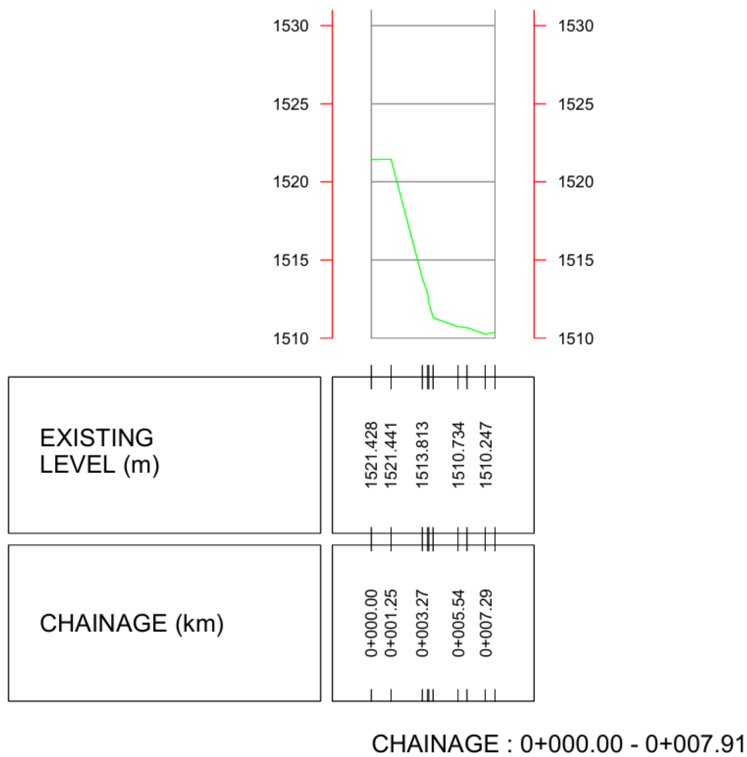

Figure 4. Cross-section view of plane 2

3.2 Kinematic failure analysis

Kinematic slope failure analyses of different rock slopes were done with the help of stereographic projection. Dips v 6.0 software was used for the kinematic failure analysis. The joint set data for kinematic analysis were taken from five different locations.

The hill slope of the landscape had a slope of 82 degrees and aspect 50 degrees (82°/50°). The orientation of the observed discontinuities had the following dip amount and dip direction: J1: 63°/50°, J2: 84°/248°, J3: 80°/27°, J4: 12°/135°, J5: 7°/345°

J1 as shown in Figure 5 is plane failure; its parallel to the hill slope with dip angle less than that of the hill slope. W1 and W2 are the wedges critical in failure in the lateral side of the hill slope.

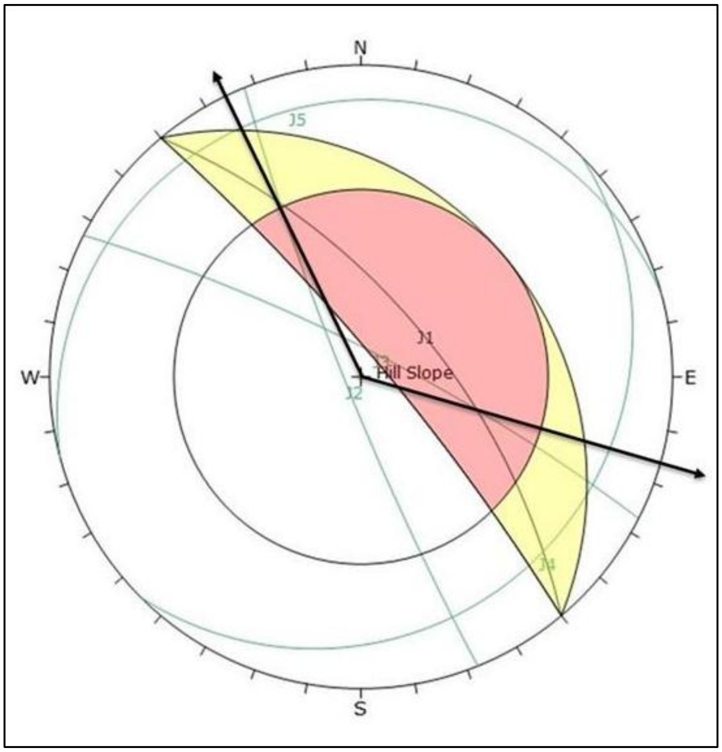

Figure 5. Stereographic plot for joint set analysis

3.3 Dynamic Cone Penetration Test (DCPT)

Five areas of critical sections were selected to obtain the graph of DCPT as shown in Figure 6.

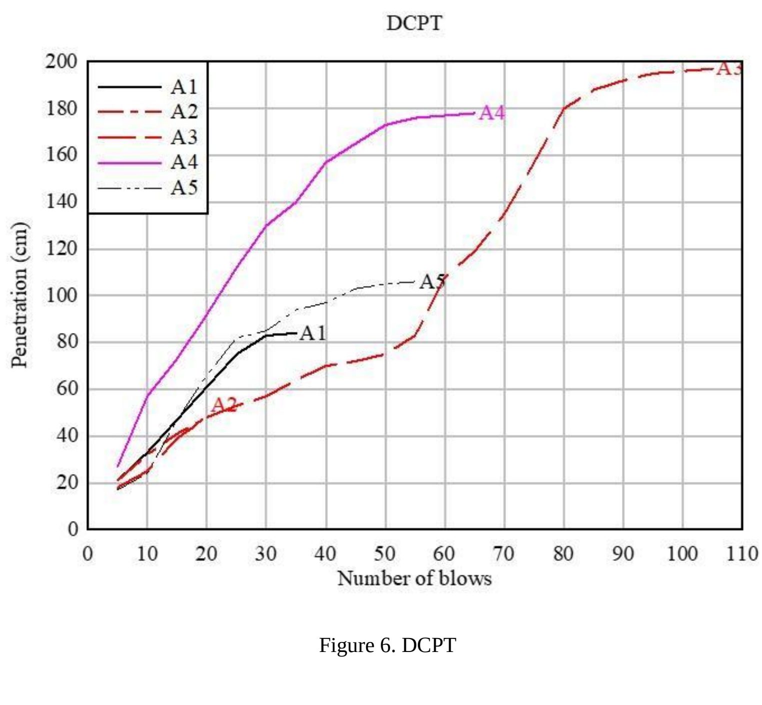

Figure 6. DCPT

3.4 Moisture content

The result of moisture content from five different samples are shown below in Table 1. The obtained values were seen in the range of 33-45%.

Table 1. Moisture Content

Sample no.	Moisture Content (w %)
A1	39.54
A2	36.50
A3	45.76
A4	34.60
A5	33.67

3.5 Grain size analysis

The grain size distribution with the fines % for each sample are presented in Table 2. Hydrometer Analysis was done only for samples A1, A3 and A5 because the percentage retained on pan was above 12 % (Das, 2002).

Table 2. Grain Size Analysis

Sample No.	Gravel %	Coarse sand %	Medium sand %	Fine sand %	Fines %	Cc	Cu
A1	8	17.6	45.6	14.8	14	3.27	33.39
A2	9.2	11.6	27.6	46	5.6	0.42	7.14
A3	4.4	5.2	28.8	46.8	14.8	1.1	7.39
A4	6.8	12	32	37.6	11.6	0.58	12.66
A5	7.2	11.2	29.6	32.8	19.2	0.35	15.06

Following classification is obtained:

A1= SM (Silty sand)

A2= SW-SM (Well graded sand with silt)

A3= SM (Silty sand)

A4= SW-SM (well graded sand with silt)

A5= SM (Silty sand)

3.6 Specific gravity

The specific gravity was determined by the pycnometer method using ASTM standard. The specific gravity of the study area ranges from 2.61-2.66. It is used for calculation of saturated unit weight. The result of specific gravity of the soil samples are given in Table 3.

Table 3. Specific Gravity

Sample no.	Specific Gravity
A1	2.661
A2	2.664
A3	2.658
A4	2.663
A5	2.607

3.7 Atterberg's limit

The calculated plasticity index percentage and liquid limit percentage values were plotted in the plasticity chart which shows the most of the soil falls below A-line and one between A-line and U-line (Figure 7). The result of Atterberg's limit test is shown in Table 4.

Table 4. Atterberg's Limit Test

Sample No.	Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)
A1	32	26	7
A2	32	28	5
A3	46	33	14
A4	33	30	2
A5	56	26	30

With the above data, the plasticity chart was obtained and referring to the ASTM standard. It was found that sample A1, A2, A3 and A4 were found as ML-OL and sample A5 was found as an outlier in the CH-OH. This may be due to the presence of organic material while sampling of soil.

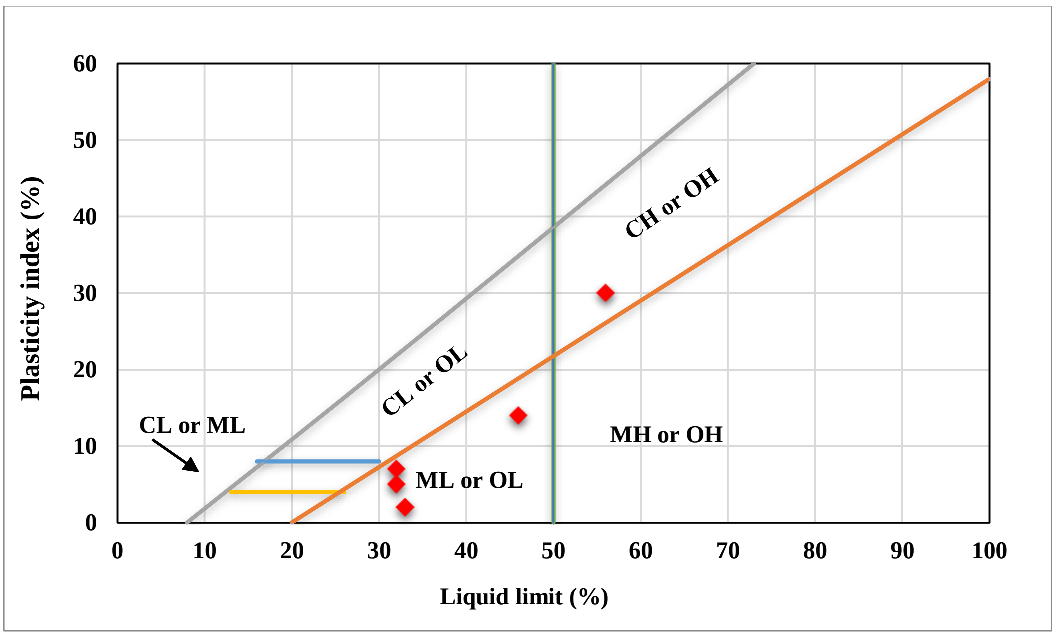

Figure 7. Plasticity Chart

The following type of soil with their respective properties are as below:

ML- Low plastic silt

OL- Low plastic organic soil

CH- High plastic clay

OH- High plastic organic soil

3.8 Direct shear test

Disturbed soil sample was used by remolding the soil from the sampling bags. The shear strength parameters obtained from the test are tabulated in Table 5.

Table 5. Direct Shear Test

Sample No.	Cohesion (kPa)	Friction angle (°)
A1	11.25	26.96
A2	11.249	26.96
A3	6.67	28.58
A4	19.31	21.73
A5	13.19	27.22

The cohesion was found to be 12.3331, slope was found to be 0.49498 and angle of internal friction was found to be 26.2896 as obtained from direct shear test.

3.9 Point load test

Three irregular rock samples were taken from the field to conduct the point load test. The observations indicate that the geological formation at our site primarily consists of phyllite within the Seti formation. Hence, the UCS value of phyllite is used in the slide software for analysis. The irregular lump test method was performed (Suryakanta, 2014). The test yielded the UCS values as shown in Table 6.

Table 6. Point Load Test

Sample ID	Sample	Width 1(cm)	Width 2(cm)	Depth, D(cm)	Load(N)	Equivalent core diameter (De^2)	Point load strength Index (Is) Mpa	UCS (qc) Mpa
1	Schist	22	13.9	7.4	12500	16912.46	0.74	11.09
2	Phyllite	13.2	11.96	6.96	17000	11148.09	1.52	22.87
3	Quartzite	11.46	9.93	6.96	40500	9477.65	4.27	64.10

3.10Bulk density test

The results of the bulk density test carried out is shown in Table 7.

Table 7. Bulk Density

Sampl e No.	Average Bulk Density(gm/cm3)	Density (kg/m3)	Average Dry Density(gm/cm3)	Dry Density (kg/m3)	Wet Unit Weight(N/m3)	Dry Unit Weight(N/m3)
A1	1.50	1502.79	1.07	1071.33	14727.34	10499.08
A2	1.26	1258.41	0.91	911.58	12332.43	8933.53
A3	1.53	1527.55	1.08	1077.86	14970.01	10563.06
A4	1.50	1500.12	1.16	1157.35	14701.17	11342.03
A5	1.34	1343.96	1.01	1013.27	13170.83	9930.00

3.11ERT profile used for slide software modeling

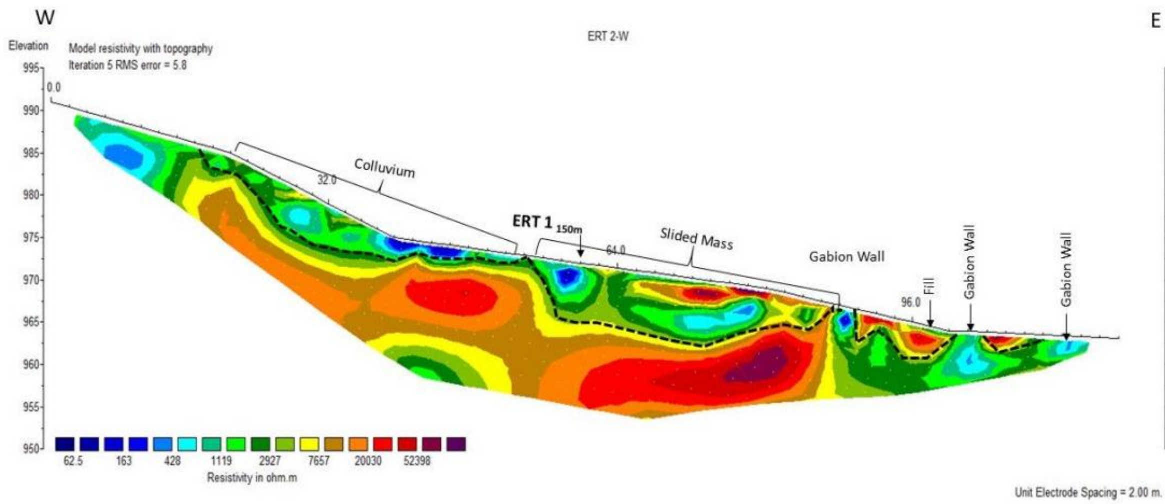

Figure 8. ERT profile (Source: GSI, 2023)

3.12Slope stability analysis and mitigation

Three different slopes were selected for modeling. The geotechnical properties of the slope needed for modeling were collected from field observation and laboratory test calculation along with ERT data observation. As shown in the contour map in Figure 3, three profiles were selected based on field observation for areas of utmost importance. The material boundary was selected suitably referring to the data from ERT graph along with possible water surface. The desired Factor of Safety for the slopes is in the range of 1.2 to 1.5 (Shiferaw, 2021) (Gunawan, Surjandari and Purwana, 2017). The information used for slope modelling are listed in Table 8

Table 8. Distribution of model materials and their representative properties

Material	Parameters	Value
Top Soil (Yellow at top)	Saturated Unit Weight (kN/m^3)	17.29
	Cohesion (kPa)	12.33
	Phi (degrees)	26.28
	Strength Type	Mohr-Coulomb
Fractured Rock (Light green at mid)	Unit Weight (kN/m^3)	20.64
	Strength Type	Generalized Hoek-Brown
	UCS (kPa)	22870
	GSI (=referring GSI chart)	21
	Intact Rock Constant	7
	Disturbance Factor	0.7
Bed Rock (Dark green at bottom)	Unit Weight (kN/m^3)	20.64
	Strength Type	Generalized Hoek-Brown
	UCS (kPa)	22870
	GSI (referring GSI chart)	40
	Intact Rock Constant	7
	Disturbance Factor	0.7

Soil nailing is considered as the best approach to achieve the required Factor of safety after performing trials with different mitigation measures. The design parameters and properties of support ((Tang and Jiang, 2015), (Jambunathan and Lal, 2022)) used on the unstable slope are shown in Table 9.

Table 9. Properties of Soil Nails used for mitigation

Mitigation Measure Adopted	Parameters	Descriptions and Values
	Force Application	Passive Method
	Out of plane spacing	2 meters
	Tensile Capacity	210 kN
Soil Nails	Plate Capacity	100 kN
	Self-weight	6 m in length, provided normal to the boundary at 3m distance between the supports
	Seismic Load (0.3 as horizontal coefficient) (NBC 105:2020)	7 m in length, provided normal to the boundary at 1 m distance between the supports

3.12.1 Slope Profiles

For slope profile 1, the stability analysis shows that Factor of Safety was found to be 1.079 showing its vulnerability. Separate analyses were done for self-weight and seismic load. After adopting soil nailing for self-weight only, the FOS improved to 1.561. But it was critical for seismic load with horizontal coefficient of 0.3 where FOS decreased to 1.151. In order to achieve similar FOS, the number of anchors were increased, spacing between two consecutive anchors were changed to 1m from 3m, and the length of the anchors were changed from 6m to 7m and FOS as 1.633 was obtained.

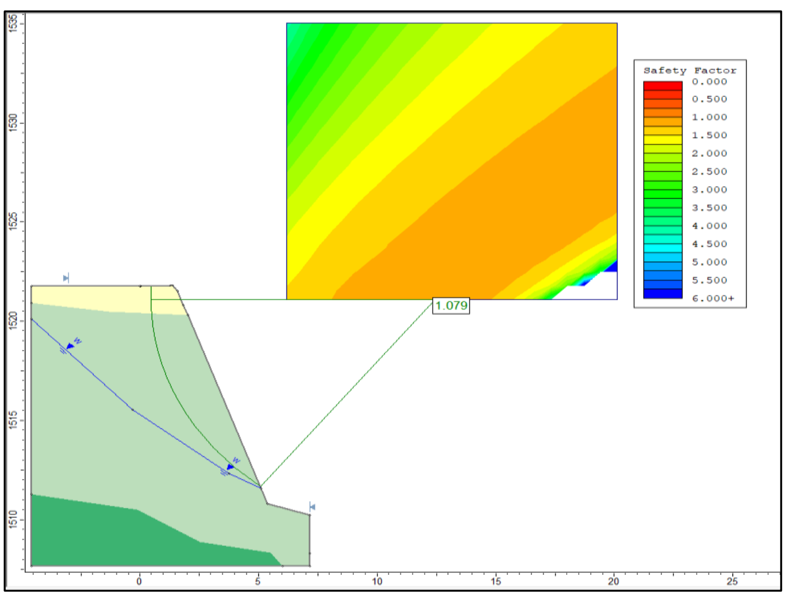

Figure 9. Profile 1

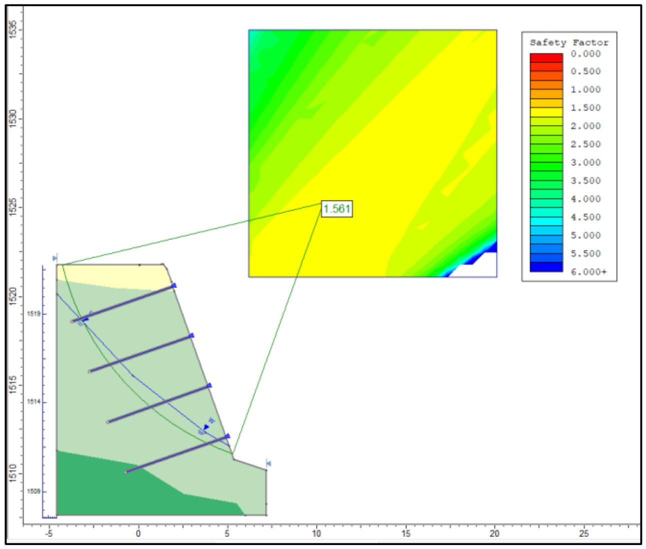

Figure 10. Profile 1: Mitigation

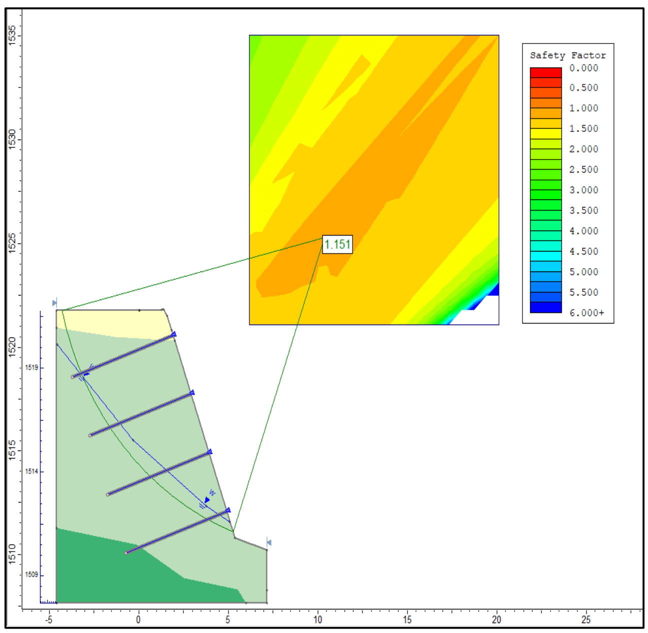

Figure 11. Profile 1: Seismic load after mitigation

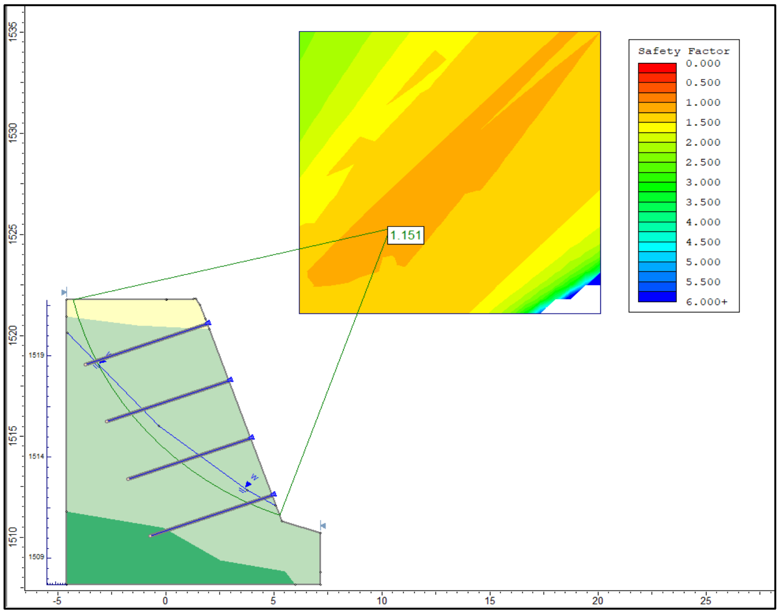

Figure 12. Profile 1: Mitigation considering seismic load

For slope profile 2: Compared to slope profile 1 and slope profile 3, slope profile 2 is concluded as the most critical slope from site observations and landslide experienced while working on the field. For this slope profile, the stability analysis shows that Factor of Safety was found to be 1.037. After mitigation, the FOS improved to 1.556 but it was critical for seismic load with horizontal coefficient of 0.3 where FOS decreased to 1.175. In order to achieve similar FOS, the number of anchors were increased, spacing between two consecutive anchors were changed to 1m from 3m, and the length of the anchors were changed from 6m to 7m and FOS of 1.573 was obtained.

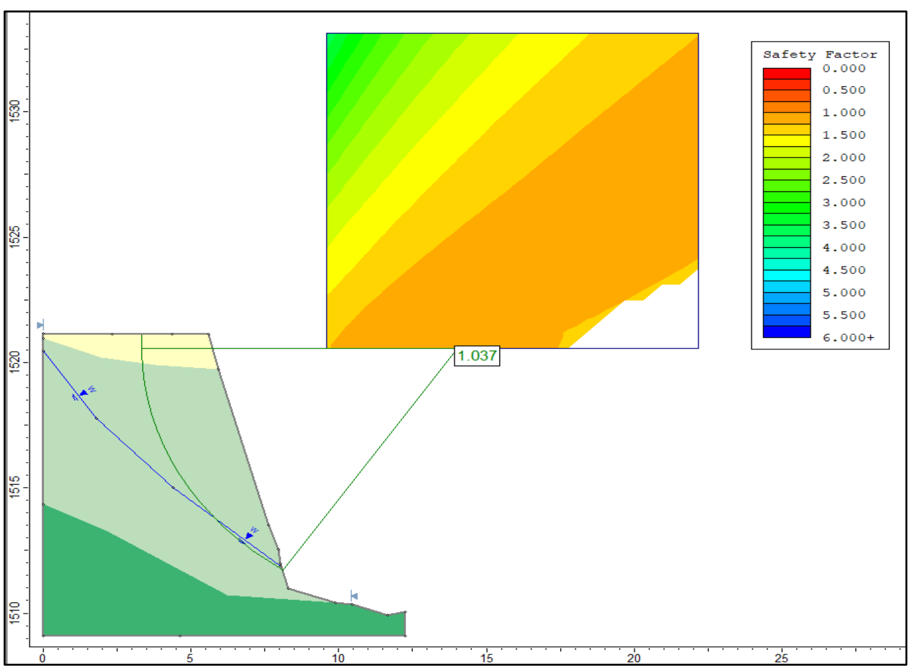

Figure 13. Profile 2

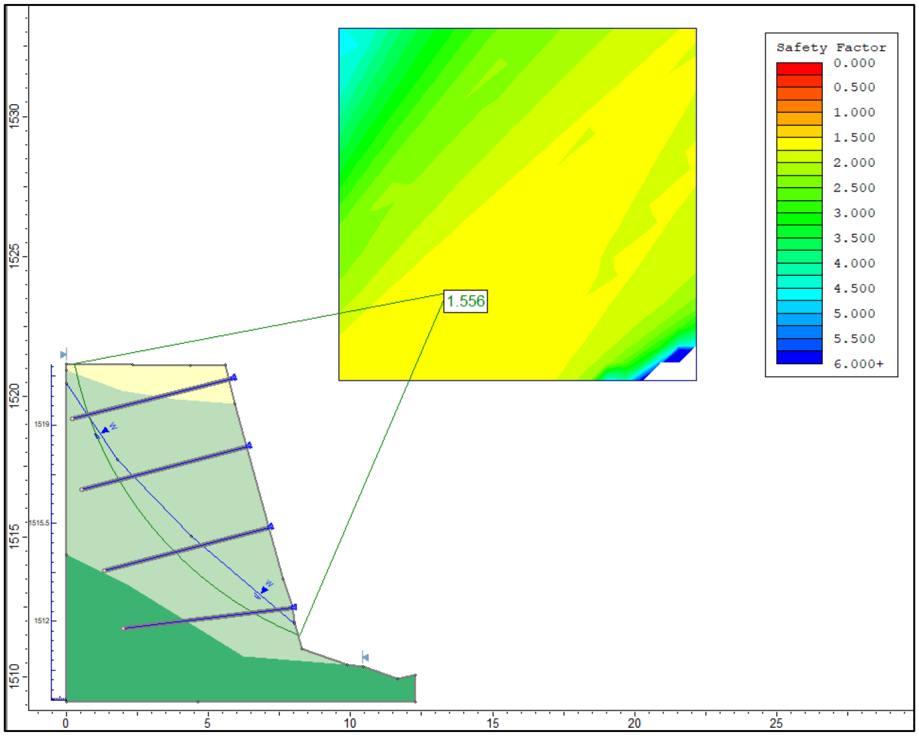

Figure 14. Profile 2: Mitigation

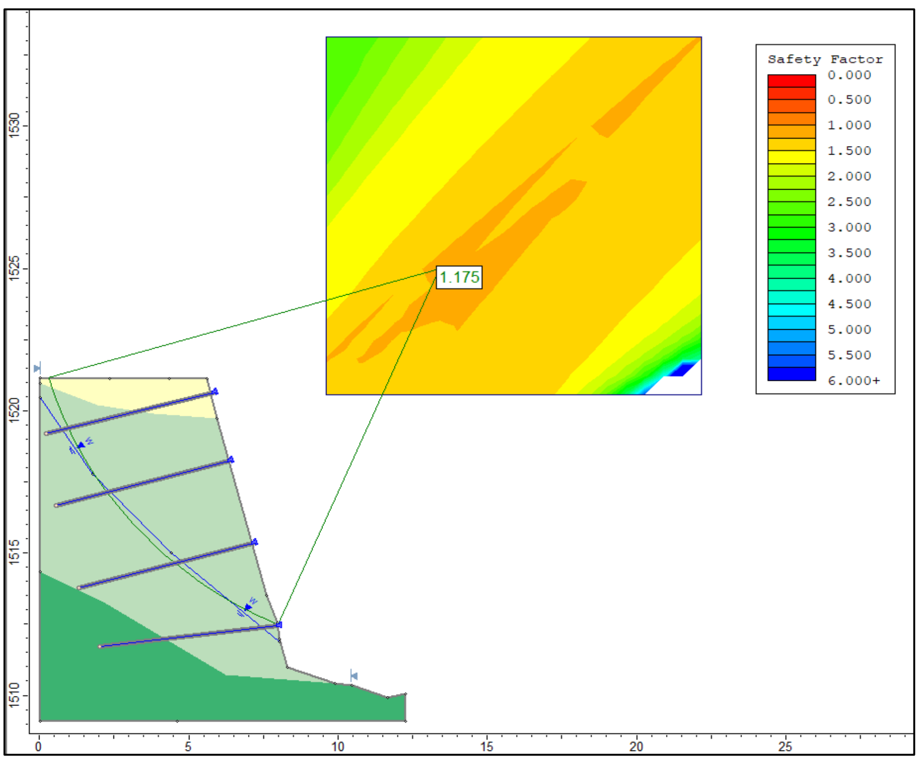

Figure 15. Profile 2: Seismic load after mitigation

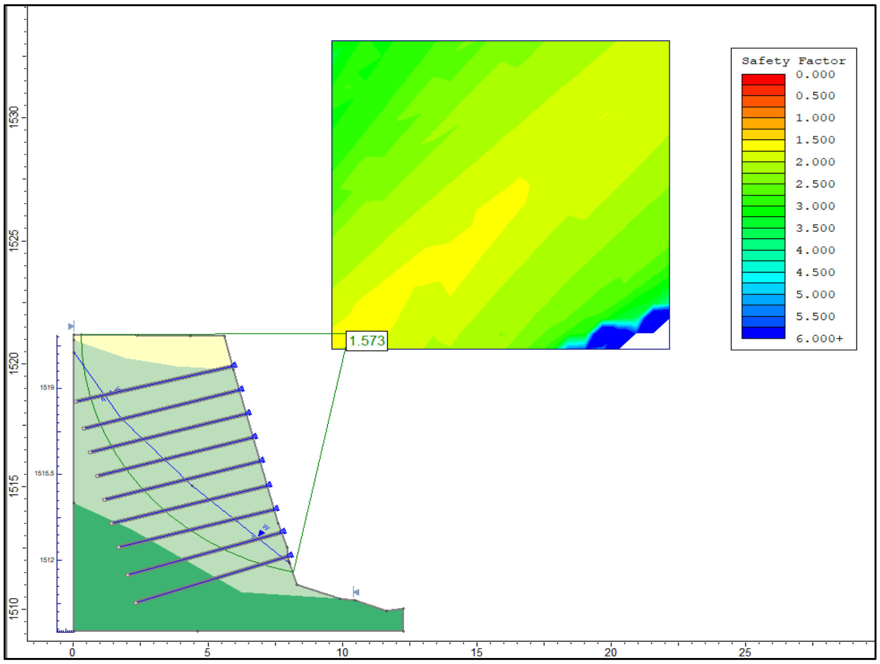

Figure 16. Profile 2: Mitigation considering seismic load

For slope profile 3, the stability analysis shows that Factor of Safety was found to be 1.001. After mitigation, the FOS improved to 1.515 but it was critical for seismic load with horizontal coefficient of 0.3 where FOS decreased to 1.147. In order to achieve similar FOS, the number of anchors were increased, spacing between two consecutive anchors were changed to 1m from 3m, and the length of the anchors were changed from 6m to 7m and FOS of 1.586 was obtained.

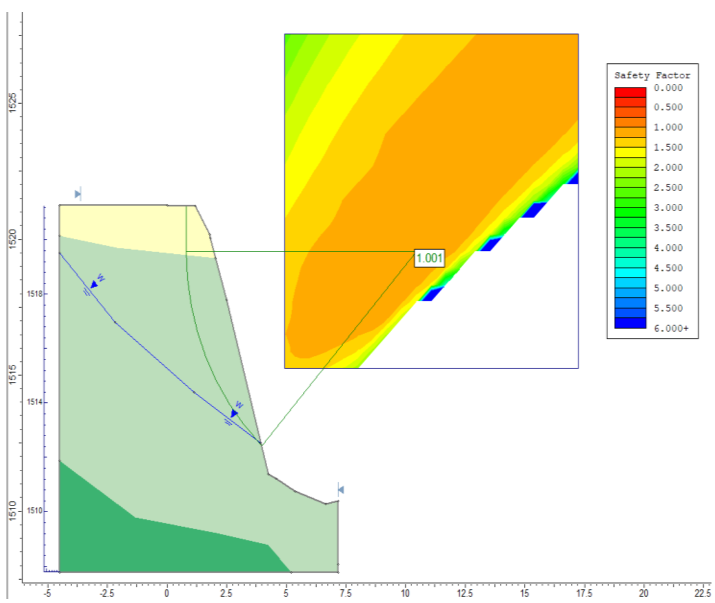

Figure 17. Profile 3

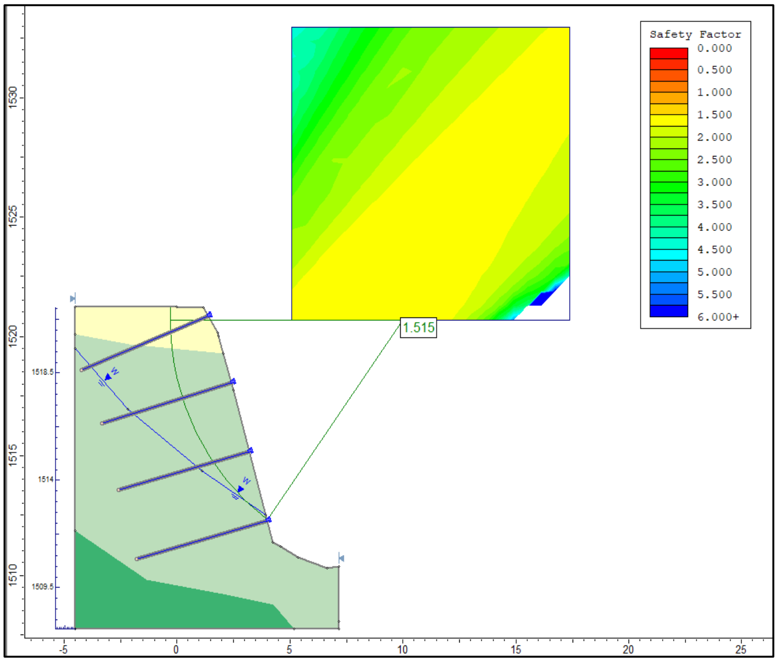

Figure 18. Profile 3: Mitigation

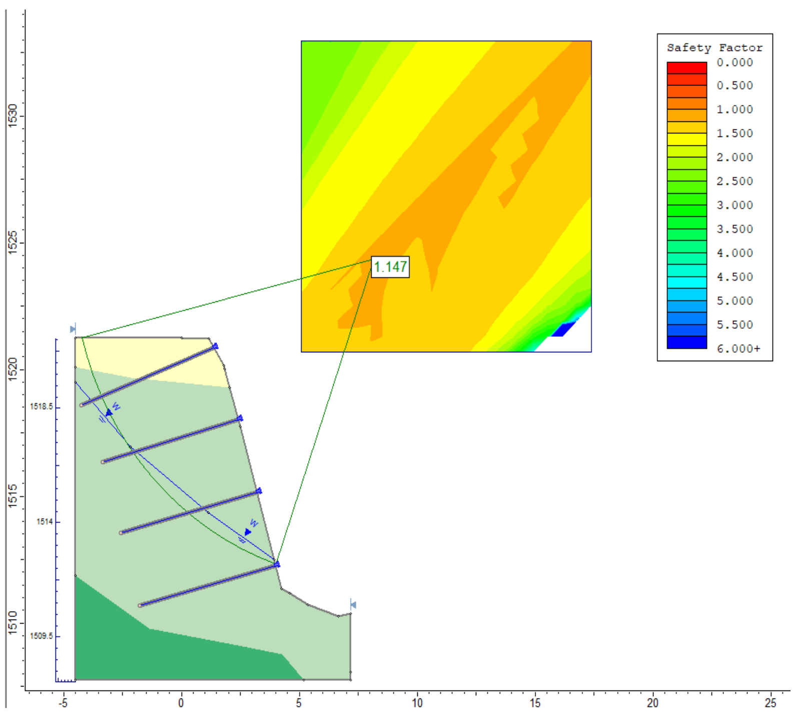

Figure 19. Profile 3: Seismic load after mitigation

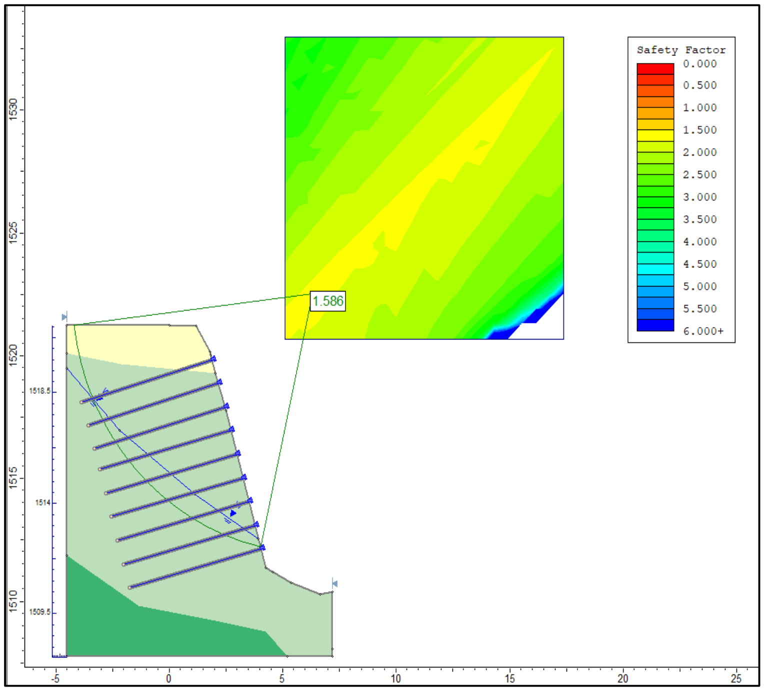

Figure 20. Profile 3: Mitigation considering seismic load

3.13 Summary of slope stability analysis

Table 10. Summary of slope stability analysis

Profiles	Factor of Safety of Basic Model	Factor of Safety with Mitigation Model	Factor of Safety with Seismic load Model	Factor of Safety Mitigation with Seismic Model
1	1.079	1.561	1.151	1.633
2	1.037	1.556	1.175	1.573
3	1.001	1.515	1.147	1.586

3.14 Validation

To validate our research on cut slope stability analysis and mitigation, which utilized the Limit Equilibrium method (LEM) and Rocscience's Slide software, we compared our findings with a test conducted for the Setikhola Hydropower Project (Panthi and Basnet, 2023). In the Setikhola study, rock slope cutting led to failure, paralleling the scenario investigated in our research. Both studies utilized LEM and Slide software for analysis.

Furthermore, in both cases, the introduction of reinforcement measures - rock bolts in the Setikhola project and soil nailing in our study - resulted in an increase in the factor of safety (FOS). This similarity in outcomes underscores the robustness and applicability of our research findings to real-world scenarios, affirming the effectiveness of the proposed mitigation strategies in enhancing slope stability. We can observe the similarities in our case with the changes in FOS from below 1.0 in unsafe conditions without mitigation to above 1.1 after mitigation.

We also compared our findings with a study conducted in Iran (Nasiri and Mohamadzade, 2022). In the Iranian study, similar to our research, the site was susceptible to landslides. They utilized the Bishop and Janbu methods, akin to our methodology, to assess slope stability along forest roads. This parallel outcome further strengthens the validity of our findings and underscores the effectiveness of employing the Bishop and Janbu methods in conjunction with Slide software for assessing and mitigating slope instability, both in forest road contexts and in similar geological settings studied about the cut-slope stability assessment along forest roads using the LEM approaches and SLIDE software.

4 Conclusion

The research concluded that the soil type of the site is low plastic silty sand and the predominantly found rock is phyllite. After observing three different profiles in varying site conditions, it was noticed that the cut slope were found to be unstable, with a Factor of Safety (FOS) near 1. The Point Load Test revealed that the Uniaxial Compressive Strength (UCS) value of phyllite, both fractured and bedrock, was constant. The Geological Strength Index (GSI) value seemed to play an important role in the slope's stability, along with the flow of water. After the application of soil nails, the cut slopes were stable, and the FOS was within the required range. However, during seismic loading, the slope profiles became critical again. By increasing the anchor length from 6 meters to 7 meters and decreasing the distance between supports from 3 meters to 1 meter, the FOS was obtained within the safe range.

Drainage issues have been identified in the field, and thus, further analysis of the site can be conducted to address these problems. Additionally, the LEM method can be utilized for slope stability and incorporated into sustainable urbanization planning in cut-slopes in Nepal, with safety being a top priority.

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