

Journal of Geological Research http://ojs.bilpublishing.com/index.php/jgr-a



ARTICLE Engineering Geological and Geotechnical Studies of Taprang Landslide, West-central Nepal: An Approach for Slope Stability Analysis

Ashok Sigdel^{*} Radha Krishna Adhikari

Department of Geology, Tri-Chandra Multiple Campus, Tribhuvan University, Kathmandu, Nepal

ARTICLE INFO	ABSTRACT			
Article history Received: 30 July 2020 Accepted: 31 August 2020 Published Online: 30 September 2020	Detailed investigation of Taprang landslide was carried out in order the understand the surface, subsurface lithological information and physical properties of soil by using multi-disciplinary methods such as engineering geological, geophysical and geotechnical studies for the determination of factor of safety for slope stability analysis. Geological study was			
Keywords: Taprang Landslide Electrical Resistivity Tomography (ERT) Groundwater Table Limit Equilibrium Method (LEM) Factor of Safety Kinematic Analysis Slope Stability	of bedrock and its discontinuities. The geophysical survey (Electrical Resistivity Tomography-ERT) were carried out to know the electrical resistivity of soil for identifying the groundwater table and slip surface of the landslide. Geotechnical analysis such as grain size analysis, liquid limit and direct shear test were carried out in order to evaluate soil classification, moisture content, cohesion and the angle of internal friction of soil for knowing the strength the soil. These soil parameters indicate the soil is very low strength. The combination of these results were used for calculating the factor of safety (FoS) by Limit Equilibrium Method (LEM) proposed by Bishop and Janbu methods. The result of factor of safety in the Taprang landslide demonstrates that the slope become stable in drained (dry) condition, remain ultimate stage in undrained (wet) condition and finally failure occurs if applied the seismic load in both drained and undrained conditions.			

1. Introduction

Solution to the landslide. To know the slope stability, main point is to calculate the factor of safety ^[4,5]. The factor

of safety is defined as the value that dividing the shear strength by the shear stress of the soil of a given slip surface. Theoretically, if the factor of safety is above unity (1), the slope may be stable ^{[6].} Hence, slope stability analysis can be calculated based on slope geometry, external and internal loading, lithological properties and rock and soil strength parameters and ground water table condition ^[7,8]. Limit equilibrium is one of the methods to calculate the factor of safety for the slope stability of landslide ^[9-14].

The slope failure in Nepal Himalaya is due to its phys-

**Corresponding Author:*

Ashok Sigdel,

Department of Geology, Tri-Chandra Multiple Campus, Tribhuvan University, Kathmandu, Nepal; Email: ashoksigdel80@gmail.com

iographic and topographic condition, high seismicity, and torrential rainfall during monsoon season ^[15-17]. Slope failure is the major erosion process in the mountainous region of the Himalavas especially in hillslope and road cuts ^[18,19]. Several researchers have focused on the study of the landslide and slope stability in Nepal Himalava. Among them, most of these researchers studied the identification and field mapping of the landslides and debris flow ^{[20-25].} Other researchers have focused on the zonation of landslide hazard and susceptibility by GIS-based mapping such as Digital Elevation Model (DEM) and weight of evidence etc [26-28]. Rainfall data models are also taken for the study of the landslide ^[29-31]. Clay mineralogy is also become important factors to know the cause and mechanism of the landslide ^{[32,33].} Very few studies of landslide have been done by using geophysical method ^[34]. Till date, very limited researches have been focused on calculating factor of safety by Limit Equilibrium and Finite element methods [35,36]. Therefore, this study has attempted to characterize the Taprang landslide in detail as a case study by using Limit Equilibrium Method. (LEM).^[37] studied the effect and losses caused by the Taprang landslide in and around the Taprang village and Madi River area and mentioned that the Taprang village is in high risk. Recently, ^[38] studies the development of Taprang landslide and revealed that advancement histories of landslide based on geology and clay mineralogy and shown that continue movement of this landslide threaten the lives and properties of Taprang village. These studies lacking of detail investigation based on geophysical and geotechnical parameters. Therefore the main objective of this study is to analysis slope stability by combining geological, geophysical (Electrical Resistivity Tomography) and different geotechnical parameters to know the mechanism, and to determine the factor of safety adopting Limit Equilibrium Method (LEM) proposed by Bishop (1955)^[39] and Janbu (1954)^[40].



Figure 1. Location map of the study area

2. Geological Setting

The Taprang landslide is located in the Madi Rural Municipality ward no. 6 at Kaski district, Gandaki province and lies in right bank of the Madi River (Figure 1). The maximum elevation is 1650 m at its crown and the minimum elevation is 1020 m at its toe. The landslide lies within 28° 18′ 84.49″ to 28°18′ 34.24″ N and 84° 04′ 24.93′ to 84° 05′ 23.75″ E and covers an area of about 0.67 km2. The length of this huge and large landslide is about 1700m from crown to toe. Several small gullies and tributaries are flowing throughout the landslide to form major stream which is finally joins to Madi River.

Geologically, the landslide area and its periphery covers parts of the Lesser Himalayan and the Higher Himalayan Zones of the Nepal Himalaya^[41]. The Lesser Himalayan Zone comprises variegated calcareous graphitic schist with grey to white quartzite, limestone and marble. The thickness of the individual beds of schist and quartzite with calcareous rocks such as limestone and marble range from 1mm to 50 cm and 10 cm to 1.5 m respectively. Similarly, the Higher Himalayan Zone comprises high grade metamorphic rocks such as schist, quartzite and gneiss. Similarly, detail geological study ^[38] also revealed that the landslide area falls in the Benighat slate and its equivalents of the Nawakot Complex of the Lesser Himalayan Zone in central Nepal established by ^[42]. This unit is separated from the Higher Himalayan zones by the Main Central Thrust (MCT) at upper part of the crown of Taprang landslide (Figure 2). Lithological, landslide area consists of highly weathered, fractured and sheared mica and garnet bearing schist, gneiss and quartzite in the crown to toe parts respectively.



Figure 2. Generalized Geological Map of the Landslide Area (Modified from Regmi, 2017)^[38]

3. Investigations Methods

The main methodologies adopted in this study were detail engineering geological mapping, geophysical (ERT) survey and geotechnical parameters to calculate the Factor of Safety (FoS) for slope stability.

3.1 Engineering Geological Mapping

Maps of engineering geological conditions of this landslide are prepared in the scales 1:5000. Basic maps of slope failures are compiled from a study of archival materials, previous studies, using interpretation of Google earth image, field mapping. In the map, several units are shown which are relevant to the evolution of slope failures. The units include particularly colluvial residual soil, debris flow, water saturation condition, scrap, and bedrock condition etc. The field data (attitude of rocks in outcrop, geological structures) was plotted in the topographic map and thus the geological map was prepared. The strike and dip of foliation in the schist, joints, and the ground surface slope were measured near the crown, and main body part of the landslide to obtain information about rock fractures and constrain kinematic slope stability analyses. Spacing of discontinuities, roughness of discontinuities, groundwater flow, discontinuity length, aperture, or separation, infilling materials, and weathering grade were also recorded. During the survey of landslide, engineering geological parameters i.e. major landslide boundary, spring zones, gullies, forest boundary were defined. ArcGIS 10.1.4 was used for the preparation of maps.

3.2 Electrical Resistivity Tomography

The ERT measuring procedure consists of resistivity measurements along the pre-defined profiles, where uniformly placed stainless steel electrodes are used. The automated, programmed measurement is taken for all possible combination of electrodes, depending on the selected measured array^[43]. The increase in the spacing increases the depth range of research determined in this method for approximately one-fifth of the distance between the extreme electrodes. As a result of measurement, the apparent resistivity of rocks, representing the result of the entire heterogeneous, complex anisotropic layers, is determined in accordance with Ohm's law. The most commonly used measuring systems are the Wenner and dipole-dipole arrays due to good depth range and the best horizontal resolution, respectively^[44,45]. (Figure 3). The instruments used during ERT survey are Multi-function digital DC Electrical Resistivity (GD-10, Geomative), multiplex electrode converter (multi electrode switching equipment CS 60), multi core cables with each takes out at every 5 m and battery with 144 volt. The Wenner and Dipole-Diople arrays, which are sensitive to vertical structures and characterized by the maximum depth of penetration, was used. Data were acquired by Wenner and Dipole Dipole configurations using a 60-electrode cable 5 m apart. The data processing and interpretation was carried out by RES2D-INV software using a nonlinear optimization technique with a 2D inversion process. The least-squares inversion methods of inversion are used in RES2DINV program. The calculations also take into account the terrain layout including the topography in the data processing ^[46].



Figure 3. General electrodes configuration of ERT survey

3.3 Geotechnical Measurement

In order to obtain more complete interpretation and to analyze the correlation between the lithology of the landslides and determined electrical resistivity on the profile, six soil samples were collected from different locations of the crown and main body parts (S1, S2, S3, S4, S5 and S6) where manual holes were dug on the stable part of the slope in order to take undisturbed soil samples. The undisturbed soils samples taken from the holes were used for physical and mechanical purposes in the laboratory. The following tests were then carried out in the laboratory of Department of Geology, Tri- Chandra Multiple Campus and Department of Mines and Geology, Government of Nepal. The liquid limit, and grain-size distribution for each sample were determined, and direct shear tests were conducted of undisturbed samples.

3.3.1 Direct Shear Test

For the direct shear test, a specimen is placed in a shear box which has two stacked rings to hold the sample; the contact between the two rings is at approximately the midheight of the sample. A confining stress is applied vertically to the specimen, and the upper ring is pulled laterally until the sample fails, or through a specified strain. The load applied and the strain induced is recorded at frequent intervals to determine a stress-strain curve for each confining stress. Several specimens are tested at varying confining stresses to determine the shear strength parameters, the soil cohesion (C) and the angle of internal friction(ϕ), commonly known as friction angle. The results of the tests on each specimen are plotted on a graph with the peak (or residual) stress on the y-axis and the confining stress on the x-axis. The y-intercept of the curve which fits the test results is the cohesion, and the slope of the line or curve is the friction angle.

In the laboratory, direct shear test was carried under several loads condition i.e. 0.5kg, 1kg and 1.5kg of weight. The shear stress is calculated under each mentioned load from the tabulated value. The test was repeated three times with different value for force P (normal force) which is followed that the difference value for force T (shear force). The normal stress and shear stress can be calculated from following formulas.

 $\sigma = P/A$ where σ is the Normal stress, P is Normal Force and A is the cross sectional area of the samples.

 τ = T/A where τ is the shear stress, T is the shear force and A is the cross sectional area of the samples.

The obtained Normal stress (σ) and Shear stress (τ) parameters of the soil samples is plotted on the Normal stress & Shear stress to obtain the value of cohesive strength (c) and internal frictional angle (ϕ) of the soil samples (Figure 4).



Normal Stress (o)

Figure 4. Relationship between Normal stress and Shear stress ^[47]

3.3.2 Grain Size Analysis

A sieve analysis (or gradation test) is a practice or procedure used to assess the particle size distribution (also called gradation) of a granular material by allowing the material to pass through a series of sieves of progressively smaller mesh size and weighing the amount of material that is stopped by each sieve as a fraction of the whole mass. The number of sieve and respective sieve size used for gradation of soil sample. Sieve analysis of collected soil samples is carried out in the laboratory and retained soil mass percentage and percentage of soil passing through each sieve is calculated. The obtained percentage passing and sieve size, D, mm is plotted in graph. The left side of the curve shows the fine grain grained soil (clay/ silt) while a curve at the right side represent a coarse grain soil (sand and gravels). Well graded soil can be identified by its representation of particles of all sizes. On the other hand, a soil is poorly graded if it has an additional of certain particles and if it has dominancy of the same size it is known as uniformly graded soil. From the graph. For the partial distribution curve D_{10} , D_{30} and D_{60} values are noted and uniform coefficient (Cu) and coefficient of gradation (Cc) is calculated. By comparing the value of Cu and Cc and percentage retain in each size the soil is classified.

3.3.3 Atterberg Limit

Atterberg limits can be used to know the degree of firmness of the soil. In order to determine the soil behavior in respond to water content and its implication to landslide occurrence. The liquid limit is the water content where a soil changes from plastic to liquid behavior. The method used for the liquid limit is Casagrande method. A paste of soil and water is put in a shallow cup, the paste is cut into two parts with a deep groove and the cup is then dropped repeatedly in a standard manner until the groove has closed owing to the flow of the paste. Here only liquid limit was calculated due to presence of sandy to gravelly soil in the landslide and plastic limit is negligible.

3.3.4 Unit Weight

The unit weight is the weight per unit volume of a material. The symbol of unit weight is γ Bulk unit weight is a measure of the amount of solid particles in addition to the water per unit volume present in the soil. It is calculated as follows,

$$\gamma_t = \gamma = \frac{(W_S + W_W)}{(V_S + V_V)}$$

Saturated unit weight is equal to the bulk unit weight of soil when the total voids are filled up with water i.e. at 100% saturation.

3.4 Slope Stability Analysis

Due to complex nature of the Taprang landslide, it is impossible to conduct a single overall slope stability analysis. Therefore, kinematic analysis was used to establish the nature of rockslides in schist and quartzite near the main body part of the landslide, which may have played a key role in triggering the soil slide. The limit equilibrium Method (LEM) slope stability analyses of soil was performed using the commercial slope stability software Rocscience Slide 6.0. The analysis was performed by using two stability analysis method i.e. Bishop and Janbu method. In Limit equilibrium Method, colluvium deposits of slope between slip surface and upper layer of the ground is divided into n slices (Figure 5). Weight (Wi), the horizontal force such as earthquake (seismic load) (0), the horizontal and vertical load (Oxi.), the water pressure (*Ui*), the interslice forces (Ei, Xi, Ei+1, Xi+1), the normal force (Ni), and the sliding force (Ti) of the slip surface are the overall forces acted on the *i*th slice ^[48]. The factor of safety is calculated as:

 $\mathbf{F}_i = \frac{N_i \tan \varphi_i + c_i b_i \sec a_i}{T_i}$

Where, *ci* is the shear strength parameters of the slip surface and *bi* the width of the slice.



Figure 5. Method of interslice along the slope based on $LEM^{[48]}$

Bishop's simplified method (BSM) considers the interslice normal forces but neglects the interslice shear forces. It further satisfies vertical force equilibrium to determine the effective base normal force (N'). This method satisfies moment equilibrium for FOS and vertical force equilibrium for normal force (N). Simply this method considers interslice normal force and mostly applied for circular shear failure. Janbu's simplified method (JSM) is based on a composite slip surface and the FOS is determined by horizontal force equilibrium. As in BSM, the method considers interslice normal forces (E) but neglects the shear forces (T). This method satisfies both force equilibrium and does not satisfy moment equilibrium. This method considers interslice normal force and commonly used for composite shear surface.

The input parameters for slope stability analysis depend upon slope geometry and soil and rock characteristics. The friction angle, cohesion and unit weight for soil is adopted from laboratory test. Similarly, the depth of slip surface and ground water table is taken from the ERT survey. The seismic coefficient i.e. PGA value 0.5 horizontal and 0.3 vertical is taken from the study ^[49]. The rock type and the slope geometry i.e. slope angle, elevation at toe and elevation at crown which were measured in field used to calculate slope height.

4. Results

4.1 Engineering Geological Study

A detailed engineering geological map of the Taprang landslide was prepared covering its periphery (Figure 6). The landslide comprises mainly of thick colluvium deposit and residual soil. The studied Taprang landslide is a complex type of slide consisting of the combination of several types of slides such as slide, fall and flow. Few other small scale slides are also present with in this landslide. The crown is mainly extended towards left flank of the landslide where mainly residual soil is present. The residual soil and the colluvium are the main lithology at the crown area. A large scrap is present at this places. Damaged school building, and small temple with cracked walls are present in the crown area (Figure 7 A). The rotational movement of the crown part is indicated by the tilted trees (Figure 7 B). The slope angle of the crown to main body part is about 32°. The geomorphology of the landslide from top crown part to toe part is quite distinctive due to variation of slope, deposited materials, rock exposures, rock cliff, and degree of weathering and nature of slide. Several cracks are also present at the crown part., which are aligned parallel to the main scarp. Numerous small soil slides, patches, undulating surface was observed. Water can easily flow through such cracks, thus facilitating the expansion of the landslide as well as weathering of rocks. The length of the cracks ranges from 1-3 m and depth up to 1 m by visual inspection.

The main body part of the landslide is covered by thick colluvium which consists of pebbles, cobbles and boulders of schists, and quartzite. The size ranges from 1-5 mm in diameter. Several tensional cracks and vertical cracks are also observed throughout the middle part of the landslide area. The length of the cracks ranges from 1-5 m and depth up to 2 m by visual inspection. Bedrock of schist are visible in the outcrop in some places which is highly

weathered, fractured and jointed and attached to the colluvial mass. The slope angle of the middle part of the slide is about 10°. A scarp of the landslide with bedrock is observed just end of the middle part of the landslide where mainly highly weathered schist and quartzite is present (Figure 7C). Some rock fall debris is also present in this area (Figure 7D). The slope is about 40° towards toe area. Several seepage zones are observed within the upper and middle part of the area which may be due to the percolation of water from hill slope. The land use of the main part of the slide is grassland and forest in right flank of the landslide. At the left flank, paddy and cultivated land is present and soil is loosen by the infiltration of water.



Figure 6. Detail Engineering Geological map of the Taprang landslide and its periphery

At the toe of the landslide, mainly colluvium deposits is present which is carried by the small drainages from crown part and active soil slides along the left flank of the landslide. The width of the toe of landslide is more than 100m from earlier failures consisting predominantly of coarse granular soil with some boulders and cobbles (Figure 7E). The slope angle is about 5-10°. On the other hand, weathered and fractured graphitic schist is exposed at the other side of the landslide.



Figure 7. Several field features of the Taprang landslide.
(a) Damage wall of temple located at Crown part; (b) Tilted trees showing the active rotational movement at crown;
(c) Large soil and rock scrap at middle main body part d) highly weathered fractured bedrock with rock debris at middle part; (e) Overall landslide featured observed from the toe part

4.2 Electrical Resistivity Tomography

A total of seven ERT profiles were carried out along and across the Taprang landslide which are described as following paragraphs. The location of the ERT profiles are shown in Figure 5. Zones of relatively low resistivity within the body of landslide are described with content of clayey, silty and sandy materials with water saturation and high resistivity value indicates either dry boulders and gravels and bedrocks.





Figure 8. Electrical resistivity tomography results: a) Across the landslide at crown; b) Along the landslide movement at crown; c) Across the landslide at body part; d) Along the landslide at body part; e) Along the landslide at body part; f) Along the landslide at left flank of the toe part g) Along the left flank of landslide at toe part

In the crown part, ERT profile 1 and 2 (across and along) show the distinct three layers from surface to bottom (Figures 8a, 8b). Upper Layer of the crown part consists of dry to partly saturated colluvium which have the resistivity value ranges from 1000 to 6000 Ω m. The depth of this colluvium is about 10 m from the surface. This soil probably originated from slide mass derived from old landslide materials. A water saturated zone with clay and sandy layer is present from depth 10-25 m from the surface of resistivity value ranges from 100-1000 Ω m. This layer marks the boundary between overburden and

bedrock. Hence slip surface is considered just below the saturation zone at depth 20-25 m from the surface. The bottom layer consists of highly fractured and weathered bedrock of gneiss which have resistivity value ranges from about 1500-6000 Qm Profiles 3, 4 and 5 were carried out in the main body part and consists of dry colluvium with resistivity value ranges from 2000-8000 Ω m. The depth of this colluvial layer varies from 10-20 m. A water saturated zone is present below 10-20 m from surface with resistivity value ranges from 50-300 Qm. This may represent the water table zone. This layer extends below depth up to 30 m and it shrinked at center and again expands towards right flank which is seen in the tomogram. At 20-30 m below surface, there is a boundary between top saturated zone and bedrock which represent the slip surface (profile 3, Figure 8c). Similarly, profile 4 shows that thick covered saturated to dry colluvial soil is present up to depth 10 m from surface. The resistivity value of this layer ranges from 700-6000 Ω m. At central part, there is a presence of boulder like rock patches which may represent bedrock of schist and quartzite which is also visualized by field observation. The resistivity value in this zone ranges from 3000-12000 Ω m. The depth of this layer exposed up to 20 m (i.e. 10-25 m) below surface. There is presence of structural discontinuities i.e. sheared zone in different places of this tomogram which is indicated by the presence of sheared bedrock of low resistivity value (100-500 Ω m) (Figure 8d). Similarly, upslope of the main body part consists of dry gravels and saturated soil as upper layer as shown in profile 5 (Figure 8e) with resistivity value 200-3000 Ω m. The depth of this colluvium extends up to 10 m from the surface. A water saturated zone is present below depth 10 m and found up to 20 m with resistivity value ranges from 70-200 Ω m. Below the water saturated zone, all three profiles show the sound to fractured and sheared bedrock of schist and quartzite with resistivity value ranges from 500-8000 Ω m (Figure 8e). The resistivity contrast between water saturated zone and below hard bedrock is considered as slip surface in the main body part found at depth 10-20 m from the surface.

In the left flank of the toe area (profile 6 and 7), a highly saturated colluvium and residual soil present at depth 5-15 m from the surface with resistivity value 20-5000 Ω m (Figure 8f, 8g). The distinct second layer consists of very low resistive, highly saturated, fractured porous zone where water is flowing throughout the layer and found below depth 10-50 m from the surface. The resistivity value of this layer ranges from 100-1000 Ω m. This layer marks the boundary of bedrock and colluvium. Below this layer, highly fractured, weathered bedrock of schist is present which has resistivity value ranges from 1000-8000 Ω m.

Based on the above result of ERT survey, it is concluded that the upper layer shows the dry sandy to gravelly soil. Groundwater table is undulating and identified at average 12 m depth from the surface. The slip surface is varied from depth 25 to 20 to 10 m from the surface in different locations from crown to main body part of the landslide. This geometry indicate the rotational and curved. The geometry and depth of the slip surface are used for slope stability analysis.

4.3 Geotechnical Result

From the results of direct shear test, the internal friction angle and cohesion of soil samples were determined. The soil samples collected from Taprang landslide has cohesion values almost all is zero because the absence of cementing materials which helps to combine soil particles tightly. The internal friction angle ranges from 34°-37°. These values indicate that the colluvial soil strength is quite low in the Taprang Landslide. The friction angle of granular soil is not a material constant, but depends on relative density and pressure level. Also friction angle of sandy soil decreases with increasing confining pressure, thus implying a circular soil failure envelope. Sand typically has negligible shear strength at zero confining stress and is generally modeled with a cohesion value of zero. The plotted graphs between Normal and Shear stress are shown in Figure 9.



Figure 9. Calculation of shear strength parameters from direct shear test

The grain size analysis shows that four samples are falls in the well-graded sand (SW) and two samples are classified as poorly graded sand (SP-GP) based on Unified Soil Classification System (USCS). The grain-size distribution curves of the soils are shown in Figure 10 and summarized in Table 1.



Figure 10. Grain size distribution curves of different soil samples

The liquid limit of soil sample i.e. S1, S2, S5 nd S6 resembles almost value ranges from 22-27% whereas the S3 and S4 are slightly greater i.e. 38%. It means S1, S2, S5 and S6 loses its shear strength and becomes liquid-like at a low water content than soil samples S3 and S4 (Table 1). The plasticity of the soil is almost negligible in all soil samples so excluded from the study.

Unit weight of the materials ranges from 17 to 20 KN/m³ for the all samples with average value is 18 KN/m³ considered for the study. The calculated different geotechnical parameters for all soil samples are shown in Table 1.

4.4 Slope Stability Analysis

Two kinds of analyses were performed to better understand different aspects of the landslide. Firstly, kinematic analysis was used to establish the nature of rockslides in schist and quartzite near the main body part of the landslide. Secondly, soil slope stability analysis was performed by limit equilibrium method. Four conditions were applied in this Limit Equilibrium Method. They are drained condition (dry), drained condition with seismic

C I N	Grain Size Analysis						Atterberg Limit	Direct Shear Test			
Sample No	Gravel (%)	Sand (%)	Silt/Clay (%)	Cu	Ce	Class	Liquid Limit (%)	Unit weight (Dry) (KN/m ³)	Unit weight (wet) (KN/m ³)	Cohesion (Kpa)	Friction angle
S1	18.45	77.72	3.83	8.80	0.83	SW	22.40	19.00	20.00	0	34.40
S2	21.80	72.60	5.60	6.67	0.02	SW	22.80	17.97	19.79	0	37.14
S3	18.51	78.62	2.87	9.30	0.85	SW	38.24	17.59	19.71	0	35.33
S4	6.44	87.36	6.20	5.25	0.67	SP	37.76	14.52	17.04	1.8	37.58
S5	3.54	93.58	2.88	4.72	0.80	SP	27.80	15.97	18.46	0	34.40
<i>S6</i>	29.28	67.64	3.08	11.50	0.21	SW	24.43	18.84	21.20	0	37.13

Table 1. Results of several Geotechnical Parameters

loading, un-drained condition (wet) and un-drained with seismic loading. The circular slip surface which is indicated by the geophysical Method (ERT) survey was taken. The following geotechnical parameters are considered for the stability analysis (Table 2).

Table 2. Several Geotechnical parameters	adopted	for
slope stability analysis		

Geotechnical parameters	Values		
Cohesion (soil)	0kpa		
Friction angle (soil)	36° (average)		
Unit weight (soil)	18KN/m3 (average)		
Groundwater table depth (ERT)	Below surface (12 m) (Dry), surface (wet)		
Slip surface (average), circular (ERT)	25m		
Landslide length (section)	1700m		
Seismic load (Thapa and Wang, 2013)	0.5-h , 0.3-v (gal)		

4.4.1 Rock Slope Stability

The analysis is actually the relationship of attitude of the natural hill slope with the attitude of discontinuities (joints and bedding planes). For natural hill slope, slope aspect and slope angle are taken into account whereas for discontinuities dip direction and dip angle are considered. The friction of the rock mass is also the area of interest in this analysis. The analysis is performed with aid of stereographic projection of the planes and lines of intersection of the planes^{[50].}

Field measurements show that a prominent discontinuity set in quartzite/ schist outcrops at the main body part of the landslide is oriented as $40^{\circ}/85^{\circ}$, $55^{\circ}/110^{\circ}$, $35^{\circ}/120^{\circ}$ and $75^{\circ}/315^{\circ}$ (dip/dip direction) in Figure 11a whereas the ground slope is oriented $55^{\circ}/105^{\circ}$, Similarly other discontinuities are $45^{\circ}/25^{\circ}$, $70^{\circ}/190^{\circ}$, $65^{\circ}/120^{\circ}$, $30^{\circ}/50^{\circ}$ and $60^{\circ}/210^{\circ}$ (dip/dip direction) whereas the ground slope is $50^{\circ}/100^{\circ}$ in Figure 11b. This indicates the slope is nearly parallel to the predominant discontinuities in that area as well as wedge are formed. Kinematic analysis of the discontinuous quartzite and schist using method proposed by ^[50] and the estimated rock friction angle suggests the slope is susceptible to rock sliding along the slope parallel discontinuities, as well as to wedge failures.





4.4.2 Soil Slope Stability

Stability analysis of the main colluvium slope at Taprang landslide was performed using the parameters in Table 2. For the dry season condition, the groundwater table was specified at the above the saturated soil and bedrock and considered at 12 m from the surface (ERT data) and except at the locations of toe of the slope where it was allowed to rise to the ground surface. The soil-bedrock contact found at depth 25 m. The calculated factor of safety for drained (dry) and drained with seismic coefficient (0.5-horizontal and 0.3-vertical) show the 1.4 and 0.67 and 1.3 and 0.63 from both Bishop and Janbu methods ranges from respectively. Hence, differences among the Janbu and Bishop methods were negligible for each case (Figure 12 and 13). Based on the result of slope stability, slope is stable under drained condition but unstable in drained with seismic load condition (Table 3).



Figure 12. Results of limit equilibrium slope stability analysis of the Taprang landslide; (a) drained conditions (dry); (b) drained dry condition with seismic load; (c) Undrained (wet monsoon season) and (d) Undrained (wet monsoon season) with seismic load using Bishop method



Figure 13. Results of limit equilibrium slope stability analysis of the Taprang landslide; (a) drained conditions (dry); (b) drained dry condition with seismic load; (c) Undrained (wet monsoon season) and (d) Undrained (wet monsoon season) with seismic load using Janbu method

Similarly, the results for the undrained condition (wet), where groundwater table supposed to be on the surface and undrained with seismic analysis show the factor of safety are 0.68 and 0.24 (Bishop method, Figure 12) and 0.64 and 0.34 (Janbu Method, Figure 13) respectively. This indicates the landslide mass is least stable in wet undrained condition and unstable in undrained with seismic load in both cases respectively (Table 3). This variation of factor of safety under various loading condition shows that the slope which was once stable could result in failure with impose of external load such as groundwater rise and seismic activities such as earthquake.

 Table 3. Factors of safety against sliding for drained and undrained conditions with seismic load

	Factor of safety (FoS)						
Method	Drained condition (Dry)	Drained condition with seismic load	Undrained condition (wet)	Undrained with seismic load			
Bishop	1.4	0.67	0.68	0.24			
Janbu	1.3	0.63	0.64	0.34			

5. Discussions

Slope stability analysis reveal that the initial saturation of the sandy soil (SW-SP) where groundwater table lies at depth about 12 (ERT data), a factor of safety of 1.4-1.3 was obtained (drained), indicating that slope is stable. If the groundwater table is much below the surface, the factor of safety increase and slope become stable. However, when seismic factors aids the slope become unstable. During the undrained condition (rainfall at monsoon season), saturation increases due to the rainfall over in the vicinity of the upper part of the sliding surface as well as groundwater table rises to surface, the factor of safety drops down rapidly and reaches a value of 0.64-0.68, which means the ultimate state, and landslide activation is almost begin. This result also consistent with the strength parameters such as friction angle and cohesion where soil is non cohesive, so it is easily moved toward the downslope. If seismic activities occurs in this stage the safety factors reach to 0.2-0.3 which could result failure of the surface and trigger the landslide. The view of the local peoples also support this result where some failures occurred during Gorkha Earthquake in 2015 even though the landslide was not so affected by earthquake as it lies far from the Epicenter. This result clearly demonstrate the positive effect of the rainfall on the stability of the slope. The decrease in the obtained SF due to the rainfall is consistent with field observations, where movements of the soil mass are observed during and after rainfalls. This result is consistent with the similar results obtained by ^[51] which shows the rapid decrease in factor of safety with increasing saturation in the upper part of the sliding surface.

The slope failure of the Taprang landslide began with the rotational movement in soil which is also known as creep in the upper part at crown. The rotational movement is depicted by the result of ERT data where slip surface is rotational and curved. Several recent scarps and tension cracks are also aids the expansion of the upper part and left flank of the landslide especially in the monsoon season. The colluvial soil (SW-SP) will be saturated by rainfall alone in monsoon season and continues develop the several local scraps throughout the crown part. This initiates the slope unstable during high rainfall condition indicated by factor of safety below 1 (Table 3). This factor of safety suggests the colluvial saturated (SP-SW) soil with gravels would have moved with increasing high pore-water pressure. Besides undrained (wet season) condition, seismic activities also play significant role to generate future movement of landslide as suggested by factor of safety in both the drained and undrained condition. These results indicate the dry season earthquake (drained) may somewhat initiate the landslide while wet season earthquake (undrained) could intensify the further movement resulting slope failure. The ERT, field mapping and grain size analysis reveal that presence of deformed sandy colluvial and residual soil at crown part, fractured, sheared and hard bedrock of schist and quartzite (rigid) at the main body part and debris deposits at toe parts of the landslide. This type of lithological variation suggests the movement of Taprang landslide caused by the process stretching in upslope (creep) and of shortening in the down slope. The mechanism of landslide movement was discussed by [52] [53]. Besides this, the several rock mass discontinuities favors the rock fall movement along the plane failures and wedge failures. Hence combination of both creeping process at the crown part and development of several scraps along the left flank as well as rock slides at main body part are the major causes of slope instability and enlargement of the Taprang landslide. So it is highly recommended to adopt appropriate mitigation measures to control of such failure of the Taprang landslide immediately to save lives and properties nearby the landslide.

6. Conclusions

The geological, geophysical and geotechnical studies revealed the variation in lithology and morphology in different part of the Taprang landslide. Engineering geological study show the main lithology consists of colluvium mass mainly consists of sandy soil, residual soil and pebbles, cobble and boulders in the upper part. The lithology mainly consists of fractured, weathered schist and quartzite at the main body part of the landslide and more than 100 m width debris are deposited in the toe part of the landslide. Electrical resistivity study revealed that the zones of sandy soil with gravely layers, highly saturated silty soil and bedrock of schist, quartzite and gneiss. The low resistive saturated soil and high resistive bedrock marks the boundary slip surface of the landslide and depth varies from 25-10 m from the surface. The groundwater table lies below the depth 12 m. The results of the laboratory tests showed the soil is well graded sand to poorly graded sand (SW-SP) with low cohesion. The calculated friction angle varies from 34°-37°. The liquid limit ranges from 22%-37% and not very consistent. Slope stability analysis based on Limit Equilibrium Method show the factor of safety of 1.4-1.3 in drained (drv) condition in both the Bishop and Janbu method, indicating slope is stable in this stage. However, when seismic load is inserted which aids the slope become unstable. During the undrained (rainfall at monsoon season), saturation increases, the factor of safety drops down rapidly and reaches a value of 0.6, which means the ultimate state, and landslide activation will began. If seismic activities occurs in this stage the factor of safety reaches to 0.2-0.3 which could result slope failure. The results clearly demonstrate the positive effect of the rainfall on the stability of the slope in the upper crown part. The presence of discontinuities in bedrock of schist and quartzite at main body part show the plane and wedge failure along the natural slope which causes the rock slides. These weak bedrocks acts as another causes for the future expansion of the landslide. Finally, geological, geophysical (ERT) coupled with geotechnical parameters are useful tools for the characterization of landslides and determine the factor of safety for slope stability analysis. This study can be used for designing the proper mitigation measures of the Taprang landslide in future.

Acknowledgement

We would like to thank Department of Geology, Tri-Chandra Multiple Campus for providing support for this M.Sc. dissertation work to second author. We are equally thankful to Department of Mines and Geology (DMG) for providing necessary laboratory test. We are also indebted to Mr. Bibas Parajuli and Mr. Suvas Acharya for helping in fieldworks and preparing some images during manuscript preparation.

References

 Abramson, L. W., Lee, T. S., Sharma, S., Boyce, G. M. Slope Stability Concepts. Slope Stabilization and Stabilization Methods, Second edition, published by John Willey & Sons, Inc., 2002, 329-461.

- [2] Taylor, M. J., Burns, S. F. Slope and seismic stability of Castle Lake Debris Dam, St. Helens, Washington. Landslide and Avalanches: ICFL 2005 Norway, Edr. Senneset, Flaate and Larsen, 2005.
- [3] Thielen, A., Friedel, S., Plotze, M., Springman, S.M. Combined approach for site investigation in terms of the analysis of rainfall induced landslides. Proceedings of the 16th Int. Conf. on Soil Mechancis and Geotechnical Engineering, Osaka Japan, 2005: 2591.
- [4] Chen N Z., Y., Shao C. M. Evaluation of minimum factor of safety in slope stability analysis. Can. Geotech. J., 1988, 25: 735-748.
- [5] Zheng, H., Tham, LG., Liu, D. On two definitions of the factor of safety commonly used in the finite element slope stability analysis Computers and Geotechnics, 2006, 33, (3): 188-195. https://doi.org/10.1016/j.compgeo.2006.03.007
- [6] Park, S., Kim, W., Lee, J., Baek, Y. Case Study on Slope Stability Changes Caused by Earthquakes-Focusing on Gyeongju 5.8 ML EQ. Sustainability, 2018, 10(10): 3441. https://doi.org/10.3390/su10103441
- [7] Ban, L., Du, W., Qi, C. Stability analysis of anchored slopes based on a peak shear-strength criterion of rock joints. Environ Earth Sci, 2020, 79: 215. https://doi.org/10.1007/s12665-020-08961-0
- [8] Karaman, K., Ercikdi, B., Kesimal, A. The assessment of slope stability and rock excavatability in a limestone quarry. Earth Sci. Res. J. [online], 2013, 17: 169-181.
- [9] Ali, J., Ahmed, K., Akhtar, K., Qudooskhan, A., Hussain, M. Stability analysis of slopes using numerical simulation based on finite element method and limiting equilibrium approach. AARJM, 2015, 2 (3): 371-379.
- [10] Deng, G., Xu, T., Chen, RNumerical Analysis on Stabilizing Mechanism of Soil Nails in Steep Fill Slopes Subjected to Rainfall Infiltration Using a Hypoplastic Model. Arab J Sci Eng., 2018, 43: 5079-5090. https://doi.org/10.1007/s13369-017-2937-9
- [11] Lin, H., Zhong, W., Xiong W., Tang, W. Slope Stability Analysis Using Limit Equilibrium Method in Nonlinear Criterion. The Scientific World Journal volume, 2014, Article ID 206062, 7 pages . http://dx.doi.org/10.1155/2014/206062
- [12] Liu, S.Y., Shao L.T., Li H.J. Slope stability analysis using the limit equilibrium method and two finite element methods. Computers and Geotechnics, 2015, 63: 291-298.
- [13] Mandal, J., Narwal, S., Gupte, S.S. Back Analy-

sis of Failed Slopes - A Case Study., International Journal of Engineering Research & Technology (IJERT),2017, 6(5).

[14] Sarkar, S., Samanta., M., Sharma M., Dwivedi., A. Slope Stability Analysis and Suggestive Measures for an Active Landslide in Indian Himalaya. In book: Recent Research on Engineering Geology and Geological Engineering, 2019.

DOI: 10.1007/978-3-030-02032-3_1

- [15] Adhikari, DP., Koshimizu, SDebris flow disaster at Larcha, upper Bhotekoshi Valley, Central Nepal. Island Arc, 2005, 14(4): 410-23.
- [16] Dhital, MR. Causes and consequences of the 1993 debris flows and landslides in the Kulekhani watershed, central Nepal, 2003.
- [17] MoHA and DPNet, Nepal Disaster Report 2015. Ministry of Home Affairs (MoHA) and Disaster Preparedness Network-Nepal (DPNet): 2015, Kathmandu.
- [18] Barnard, PL., Owen, LA., Finkel, RC. Quaternary fans and terraces in the Khumbu Himal south of Mount Everest: their characteristics, age and formation. J. Geol. Soc. Lond., 2006, 163: 383-399.
- [19] Selby, MJ. Landforms and denudation of the High Himalaya of Nepal: results of continental collision. Z Geomorphol Neue Folge, 1988, 69:133-152. (Supplemented)
- [20] Dhital, MR., Upreti, BN., Dangol, V., Bhandari, AN., Bhattarai, TN. Geological engineering methods applied in mountain road survey: an example from Baitadi-Darchula Road Project, Nepal. J Geol Soc of Nepal, 1991, 59: 89-94.
- [21] Gallo, F, Lavé J. Evolution of a large landslide in the High Himalaya of central Nepal during the last half-century. Geomorphology, 2014, 223: 20-32.
 DOI: 10.1016/j.geomorph.2014.06.021
- [22] Ibetsberger, HJ. The Tsergo Ri Landslide: An uncommon area of high morphological activity in the Langthang Valley, Nepal. Tectonophysics,1996, 260(1): 85-93.
- [23] KC, J., Gautam, D., Neupane, P., Paudyal, KR. Landslide inventory mapping and assessment along the Ramche-Jharlang area in Dhading, Rasuwa and Nuwakot districts, Lesser Himalaya Central Nepal. Journal of Nepal Geological Society, 2018, 55 (Sp. Issue): 103-108.
- [24] Timilsina, M., Bhandary NP., Dahal RK., Yatabe, R. Distribution probability of large scale landslides in Central Nepal. Geomorphology, 2014, 226:236-248.
 DOI: 10.1016/ j.geomorph.2014.05.031
- [25] Upreti, BN., Dhital MR Landslide studies and management in Nepal ICIMOD, Nepal. 1996: 87.

- [26] Acharya, T., Yang, I. T., Landslide Hazard Zonation using GIS: A Case Study from Sindhupalchowk, Nepal, International Journal of Applied Engineering Research, 2015, 10 (7): 18385-18394.
- [27] Devkota, KC. Regmi A.D., Pourghasemi, H.R., Yoshida, K., Pradhan, B., Ryu. IC., Dhital, MR., Althuwaynee O. F. Landslide susceptibility mapping using certainty factor, index of entropy and logistic regression models in GIS and their comparison at Mugling-Narayanghat road section in Nepal Himalaya. Nat Hazards, 2013, 65:135-165. DOI: 10.1007/s11069-012-0347-6
- [28] Poudel, K., Regmi, AD. Landslide susceptibility mapping along Tulsipur-Kapurkot road section and its surrounding region using bivariate statistical model. Journal of Nepal Geological Society, 2016,

DOI: 10.3126/jngs.v50i1.22868

50(1):83-93.

- [29] Dahal, RK., Hasegawa, S. Representative rainfall thresholds for landslides in the Nepal Himalaya. Geomorphology, 2008, 105(3-4): 429-443.
- [30] Dahal, RK., Hasegawa, S., Nonomura, A., Yamanaka, M., Dhakal, S., Paudyal, P. Predictive modeling of rainfall-induced landslide hazard in the Lesser Himalaya of Nepal based on weights-of-evidence. Geomorphology, 2008, 102: 496-510.
- [31] Bhandary, NP., Yatabe, R., Dahal RK., Hasegawa, S., Inagaki, H. Areal distribution of large-scale landslides along highway corridors in Central Nepal. Georisk: Assessment and Management of Risk for Engineered Systems and Geohazards, 2013, 7: 1-20. DOI: 10.1080/17499518.2012.743377
- [32] Regmi, AD., Yoshida K., Dhital, MR, Devkota, K. Effect of rock weathering, clay mineralogy, and geological structures in the formation of large landslide, a case study from Dumre Besei landslide, lesser Himalaya Nepal. Landslides, 2013, 10: 1-13. DOI: 10.1007/s10346-011-0311-7
- [33] Regmi, A D., Yoshida, K, Dhital, MR, Pradhan, B. Weathering and mineralogical variationin gneissic rocks and their effect in Sangrumba landslide, East Nepal. Environ Earth Sci, 2014, 71: 2711-2727. DOI:10.1007/s12665-013-2649-8
- [34] Panta, SR. Study of subsurface of a landslide by geophysical methods. In: Landslide hazard Mitigation in the Hindu Kush Himalaya, 2001, ICIMOD: 89-120.
- [35] Aryal KP. Slope stability evaluations by limit equilibrium and finite element methods, PhD Thesis, Norwegian University of Science and Technology, Trondheim, Norway, 2006.
- [36] Gurung, N., Haneberg, WC., Gunturi R., Datta, M. Engineering Geology and Stability of the Laprak

Landslide, Gorkha District, Western Nepal. Environmental and Engineering Geoscience, 2011.

DOI: 10.2113/gseegeosci.17.1

- [37] Khanal NR, Koirala A, Gurung SB, Kumar U, Rijal R, Sigdel S, Acharya C, Yadav UP Madi River, Nepal: landslide dam outburst flood risk management, ICI-MOD, Nepal, 2013.
- [38] Regmi, AD. Yoshida, K., Cui, P., Hatano, N. Development of Taprang landslide, West Nepal. Landslides, 2017, 14(3): 929-946.
 DOI: 10.1007/s10346-016-0752-0
- [39] Bishop, A. W. The use of the Slip Circle in the Stability Analysis of Slopes Géotechnique, 1955, 5(1): 7-17.
- [40] Janbu, N. Applications of composite slip surfaces for stability analysis. In: European conference on the stability of earth slopes, 3.Stockholm. Proceedings, 1954: 43.
- [41] Dhital, MR. Geology of the Nepal Himalaya: Regional Perspective of the classical collided orogen. Spri. Int Pub, Switzerland, 2015, 230-233.
- [42] Stöcklin, J., Bhattarai, KD. Geology of Kathmandu area and central Mahabharat range Nepal. In: Himalaya Report, Department of Mines and Geology, Kathmandu, Nepal, 1977, 86 p
- [43] Pasierb, B., Resistivity tomography in prospecting geological surface and anthropogenic objects. Techn Trans Environ Eng, 2012, 23: 201-209.
- [44] Kneisel, C., Hauck, C., Fortier, R., Moorman, B. Advances in geophysical methods for permafrost investigations. Permafrost and Periglacial Processes,2008, 31(1): 15-30.

DOI: https://doi.org/10.1002/ppp.616

[45] Loke MH, Tutorial: 2-D and 3-D electrical imaging

surveys, 2009. www.geoelectrical.com

- [46] Loke, MH. Tutorial: 2-D and 3-D electrical imagining surveys. Geotomo Software, Malaysia, 2014.
- [47] Chaulya, S.K., Prasad G.M. Slope Failure Mechanism and Monitoring Techniques Sensing and Monitoring Technologies for Mines and Hazardous Areas, 2016.
- [48] Zhao, Y., Tong ZY., Lü, Q. Slope Stability Analysis Using Slice-Wise Factor of Safety. Mathematical Problems in Engineering, 2014, Article ID 712145, 6 pages.

http://dx.doi.org/10.1155/2014/712145

[49] Thapa, DR., Wang, G. Probabilistic seismic hazard analysis in Nepal Engineering Vibration, 2013, 12(4): 577-586.

DOI: 10.1007/s11803-013-0191-z

[50] Lim, SS., Yang, HS. An analysis of plane failure of rock slopes by quantified stereographic projection. Int J of Rock Mech and Mining Sci, 2004, 41(3): 505-505.

DOI: 10.1016/j.ijrmms.2003.12.037

- [51] Stanisz, J., Pilecki, Z., Woźniak, H. Selected aspects of numerical analysis of landslide stability in Swoszowice. Geological exploration technology geothermics. Sustain Dev, 2012, 2: 77-88.
- [52] Baum, R.L., Fleming, R.W. Use of longitudinal strain in identifying driving and resisting elements of landslides. Geol. Soc. of Am. Bull, 1991, 103(8): 1121-1132.

DOI: 10.1130/0016-7606

[53] Yagi, H., Maruo, Y., Saijo, K., Nakamura, S. The September 1988 large landslide in the vicinity of MCT, Darbang, Nepal. Journal of Japan Landslide Society Tokyo, 1990, 26: 45-49.