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Slope stability issues of Bukula Landslide in Raghuganga Hydropower Project

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ABSTRACT

Slope stability concerns hold global significance among researchers, professionals, and academicians. Despite various research studies, geological and geotechnical investigations on landslides, there is still a lack of proper and comprehensive landslide hazard study, accurate data acquisition, and effective monitoring mechanism in Nepal. The objective of this present study is to identify the type of failure, causes and effects of landslide and possible mitigation measures of Bukula Landslide located within Raghuganga Hydropower Project in Myagdi. This landslide is a large-scale landslide that spans over 500 meters with vertical relief exceeding 300 m. Based on visual inspection, wedge, toppling, and buckling failures prevail within the landslide. The bedrock is thinly foliated and moderately jointed, while 3 m long tension cracks are observed along the foot trail across the landslide area, indicating that the slope is unstable. Notably, buckling failures are common in highly jointed bedrock with low RQD. Likewise, sheared zones/weak zones were observed within the landslide area. The geological survey, kinematic analysis and numerical simulations of the hillslope revealed that Bukula landslide was triggered by sheared rock mass and river toe cutting. To mitigate the problems caused by the landslide, proper support structure installation and drainage system design are necessary.

Keywords: Landslide hazard, slope failure, tension cracks**,** numerical analysis

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INTRODUCTION

Slope stability associated with rock slopes in hydropower projects are one of the highly concerning issues (Chen et al., 2015). In mountainous country like Nepal, the slope failures, landslide, debris flow and rock fall become very problematic for planners and developers (Hewitt et al., 2008). There have been different researches, geological and geotechnical investigations on landslides globally (Froude and Petley, 2018). However, these issues are less considered in comparison to other geological issues like rock squeezing, rock bursting, weak or sheared rock mass condition during tunnel excavation, hydrological issues like flood etc. (Chen, 1995). In global aspects, civil development works like construction, city development works, mining works and hydropower development works are initiated after the proper land site division and selection (Afeni and Cawood, 2013). There are strong guidelines for development works, enough budget for geotechnical investigation and public are always aware of laws and guidelines (Porter and Morgenstern, 2013). In case of Nepal, proper guidelines are still lacking and in other hand hydropower development works are resumed neglecting the slope stability issues. There are different causative factors of rock slope (Sonmez and Ulusay, 1999). The main reason could be taken as geomorphological condition like higher altitude and steep-mountain towards northern region, fragile geological condition, rainfall, seismic activity and haphazard civil development works (Giani, 1992; Duncan, 1996; Li et al., 2008, 2011; Shen et al., 2013). The consequences that are now facing are large-scale slope failures whether it could be large landslide of Jure in Sindhupalchowk or voluminous debris flow along the Melamchi River (Maharjan et al., 2021; van der Geest and Schindler, 2016). These are large-scale slope failures and could not be neglected but medium-scale and small-scale failure which affects limited area or territory or casualty are still being underestimated. Thus, the main aim of the present study is to identify the type of failure, cause and effect of slope failure as a case study of the Bukula Landslide which lies on Raghuganga Hydropower Project of Myagdi. The landslide is not situated in main hydraulic structure but it causes problem to the access road that connects the Dam-site to highway of Kaligandaki Corridor.

In this study, slope stability analysis was analyzed through Rocscience software Phase2 on the basis of generalized Hoek and Brown failure criterion (Hoek et al., 2002; Hoek and Marinos, 2007). During simulation on finite element method (FEM), the nonlinearity of stress is addressed with taking consideration of minimal confining stresses which is associated with the slope instability (Lane and Griffiths, 2000; Fu and Liao, 2010; Lyamin and Sloan, 2002; Nekouei and Ahangari, 2013). The strength parameters of rock mass for simulation was obtained from laboratory tests.

LOCATION OF STUDY AREA

The Landslide area is located near to the Bukula village, Dagnam Darmija VDC, Myagdi District, western Nepal. Geographically it is bounded by the latitudes of 28°22'00" to 28°25'10" N and the longitudes of 83°31'50" to 83°33'00" E, at the left bank of the Rahughat Khola near Lareni Talun (Fig. 1).

GEOLOGY AND GEOMORPHOLOGY

The study area lies within the Dadagaon Phyllite Formation of Nawakot Complex (DMG, 2002; Stocklin and Bhattarai, 1977). The predominant rock types are grayish green phyllite, phyllitic slate with tin seams of dolomites and quartzites (Fig. 2). The project area lies within the Dadagaon Phyllite of Upper Nawakot Group. Phyllite is well exposed along the Rahughat Khola and in the dam site area, penstock alignment and powerhouse site. The bedrock is moderately weathered, thickly to blocky foliated, moderately strong to weak gray colored phyllite. The general trend of the bedrock is 050 to 037°. Quartzite is another rock type present in the project area which is mainly exposed at the steep cliff along foot-trail from Galeshowar to Dagnam, along the penstock and the headrace tunnel alignment uphill of Mauwaphant. Greenish grey to white quartzite is generally fresh to slightly weathered, medium to high strong, and seamy to massive. The soil types in the study area are colluvial and alluvial deposits (Fig. 3).

Phyllite is the predominant rock type within the project area. It is well exposed along Rahughat Khola, its tributaries and in and around the project site. This unit is composed of light grey to grey, thinly to thickly bedded phyllite with abundant quartz veins; which is moderate to highly weathered and medium to

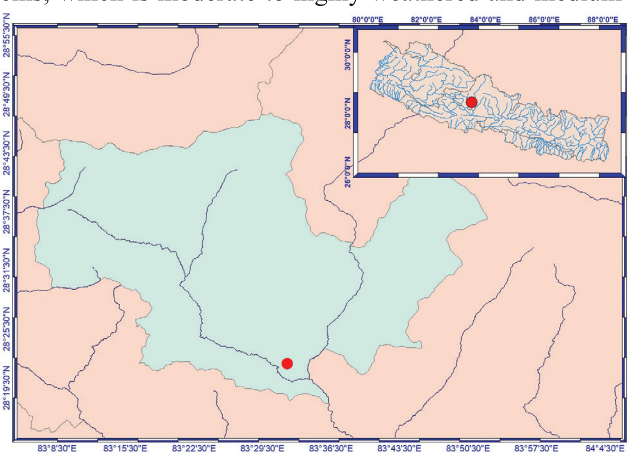

Fig. 1: Location of the Raghuganga HEP, Myagdi District of Nepal.

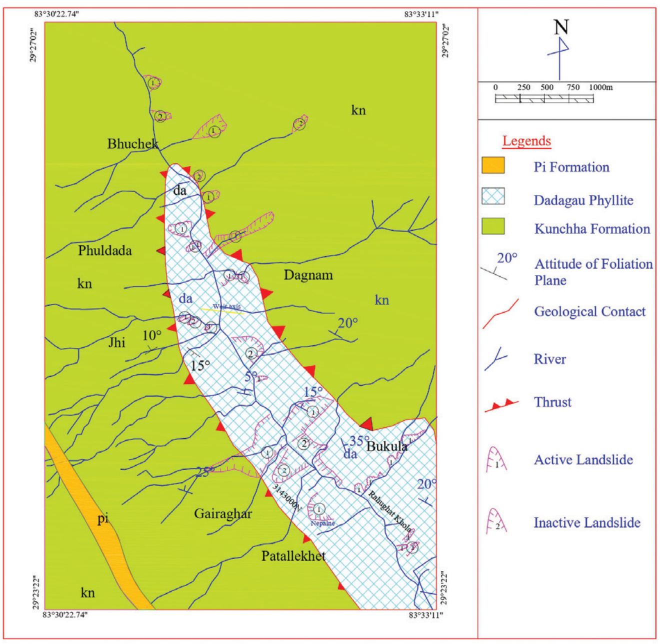

Fig. 2: Regional geological map of study area (after DMG, 2002).

less strong. Phyllite is interbanded with medium to coarse-grained, thinly to thickly bedded, grey to white and greenish grey quartzite. Black slate with carbonate beds are well exposed from the Andheri Khola to Jhi suspension bridge near head works of the project area. The slate is dark grey to black and is intercalated to thinly bedded limestone and dolomite. Quartz veins are abundant throughout this sequence. White Quartzite is mainly exposed at the steep cliffs and ridges along the foot trail from Galeshowar to Dagnam above the 1600 m elevation. Greenish grey to white quartzite is generally fresh to slightly weathered, medium to strong, seamy to massive and intercalated with phyllite. Ripple marks was also observed on the surface of the rock. Variegated quartzite with phyllite is mainly exposed from Piple to Dagnam villages. It follows the elevation ranges from 1200 to 1500 m. The quartzite is fine to medium grained, variegated, fresh to slightly weathered, medium to strong, and thinly to thickly banded which is intercalated to grey, shiny phyllite.

Rahughat Khola is east-west trending perennial snow fed river which originates from the Dhaulagiri range of the Nepal Himalaya. The gradient of this Rahughat Khola is high within the project area. Rahughat Khola passes through rugged topography with hard rock exposure and in situ residual soil. It is mainly composed of sharp crested ridges, medium to very steep slopes and gently sloping lowlands in the valley. North facing slopes are generally covered by sparse to dense mixed forest vegetation; whereas the south facing slopes are rocky with colluvial products which are partly cultivated. Many tributaries of Rahughat Khola are more or less perpendicular to it. These tributaries are moderately to deeply incised and have parallel to sub-parallel drainage patterns. Few of these tributaries flow across the proposed tunnel alignment. Three tributaries just downstream, at the right bank of the weir axis have high capacity to bring down sediments. Terraces are present at the lower stretch of Rahughat Khola.

METHODS AND MATERIAL

Kinematics analysis is applicable for finding out possible failures and determining Maximum Safe Slope Angle (MSSA) of rockslope. The founder of MSSA for blocky rockmass is Goodman (Goodman, 1976). In this study, kinematic analysis of the joint set was performed where more than 150 joint-sets were measured. However, 26 joints which most represented the ground conditions were processed utilizing a commercially available software DIPS 7.0 (Rocscience, 2017), based on equal angle stereographic projection, and major joint sets.

Fig. 3: The debris of Bukula landslide disturbing access road of hydropower project.

The simulation of the landslide was carried out based on the geotechnical investigation of the project area. Kinematical analysis is related with the motion of material only while numerical analysis is associated with the detail analysis of such force and stress responsible for motion and confirmation of results obtained from kinematical analysis. In addition, historical image of Google image was studied from 2008 to 2021. The flowchart of this study was conducted as expressed in the flowchart below (Fig. 4).

Finite Element Method is a general-purpose method that can be used to calculate many engineering interests which include the major or minor principal stress, pore pressure, displacement of rocks or soils, which are among the many characteristics of the slopes (Duncan, 1996). Phase2 which is a finite element program, is used in this study. A continuum approach of Finite Element Method is most used approach for numerical modelling of hillslope. In this approach, the rock mass is meshed applying elements of triangular or quadrilateral shape in 2D. Roller boundary conditions on either side of the slope were adopted so that they were not fixed in the vertical direction. This was done after several simulations as the boundary conditions created wall like surroundings which interfered with the analysis. The Hoek-Brown failure criterion developed by Hoek and Brown (Hoek and Brown, 1980) on the basis of Griffith theory (Griffith, 1924). The Equation 1 provides the failure of intact rock.

$$\sigma 1 = \sigma 3 + \sqrt{mi\frac{\sigma 3}{\sigma c} + 1} \qquad (1)$$

where, σc is the uniaxial compressive strength of the material and mi is a material constant which defines the brittleness of intact rock. In this equation, $\sigma 1$ and $\sigma 3$ are major and minor principal stresses respectively, that act on the rock mass and are used to define the rock mass failure envelope in the Hoek –Brown failure criterion. This criterion allows simulation of varied stress-strain behaviour from simple elastic to elasto-plastic or time-dependent creep. This method can give information about the deformations at working stress levels and is able to monitor progressive failure including overall shear failure (Griffiths and Lane, 1999).

A Geological Strength Index (GSI) assigns a numerical value between 0 and 100 to a rock mass based on the conditions of the rock mass with higher values indicating stronger and more stable conditions (Hoek and Brown, 2019). The GSI is used in

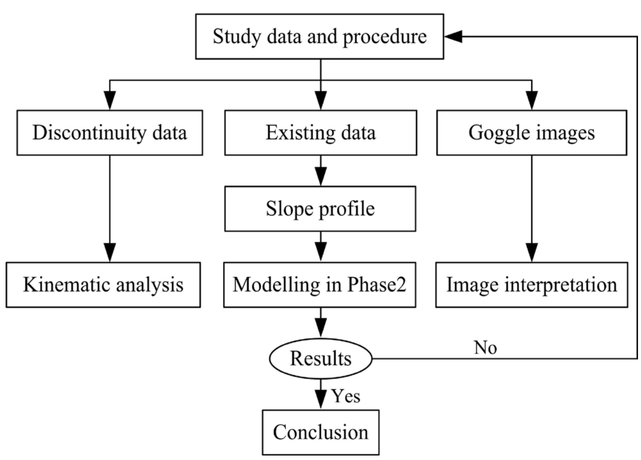

Fig. 4: Flowchart of methodology.

conjunction with other parameters such as the rock quality (Q) and the rock mass rating (RMR) to evaluate the stability of a rock mass. A modification in the GSI approach was proposed for tectonically disturbed flysch rock masses (Marinos and Hoek, 2000). In this study, the strength reduction method (SRM) was used to do this case study (Fu and Liao, 2010). The simulation was carried out in Plane Strain with maximum iteration 200 and Tolerance 0.001. The stress ratio of horizontal and vertical plane was assumed as 1. Roc lab software which was developed by Rocscience, was used to quantify the physical properties of rock mass. This activity would enable the calculation of the strength factor or factor of safety for the slope, and then the results obtained can be compared with the presented solutions from existing investigations.

Similarly, the Mohr-Coulomb failure criterion was used for simulation of colluvial deposits. It is commonly used in geotechnical engineering to predict the behaviour of soils and rocks, but it can be also used for other materials. The criterion is based on the principal of maximum shear stress. According to this principle, a material will fail when the maximum shear stress on any plane within the material reaches a critical value (Al-Awad, 2012). The Mohr-Coulomb failure criterion is expressed mathematically in Equation 2.

$$\tau = c + \sigma n * \tan(\varphi) \qquad (2)$$

where, τ is the maximum shear stress on a plane within the material, are cohesive strength of material, c, σn and φ are normal shear stress acting on the plane and the angle of friction between the particles of the material respectively.

During simulation, colluvial deposits, fractured rockmass and three types of bedrock namely quartzite, phyllite and dolomite are kept with respective physical properties for simulation. The laboratory tests of the bedrock including uniaxial compressional strength and point load test were extracted from the geotechnical investigation report of Raghuganga Hydroelectric projects of Nepal Electricity Authority for simulation (NEA, 2013, 2016 and 2018). The geotechnical properties of hill slope materials for simulation are taken as follows (Table 1).

Nature and type of landslide

Active, inactive and dormant landslides of large, medium and small sizes were depicted on the periphery of the project area. The regional geology shows that the thrust passes near to project area. The project area mainly consists of thinly foliated phyllite with low friction angle.

The landslide is large-scale slide extended up to more than 500 m of length and relief is also more than 300 m. The wedge type failure was observed based on visual inspection. The bedrock is thinly foliated and moderately jointed. The tension cracks (length 3 m) were observed along the foot-trail of landslide area, therefore, the slope is not stable. Active, inactive and dormant landslides of large, medium- and small-sizes are depicted along the bank of Raghuganga. The buckling type failure is common in highly jointed bedrock with low RQD values. Sheared zone/weak zones were observed within the landslide area. The main cause of the failure is due to fragile geological condition and thrust passes nearby the Bukula Landslide. In addition, river toe cutting, haphazard human settlement and pore water pressure are also responsible of the instability.

Table 1: Materials types and physical properties of materials.

Materials	Colluvial deposits	Fractured rock mass	Competent rock mass	Phyllite	Dolomite
Initial element loading	body force only	field stress and body force	field stress and body force	field stress and body force	field stress and body force
Unit weight (MN/m^3)	0.02	0.025	0.026	0.027	0.026
Elastic type	isotropic	isotropic	isotropic	isotropic 18000	isotropic 18000
Young's modulus (MPa)	18000	18000	20000	18000	19000
Poisson's ratio	0.3	0.3	0.26	0.3	0.3
Failure criterion	Mohr-Coulomb	Hoek-Brown	Hoek-Brown	Mohr-Coulomb	Hoek-Brown
Peak friction angle (^{o})	27	0	50	30	2.346
Peak cohesion (MPa)	9	25	2.977	5	0.0025
Material type	Plastic	0.341	0.0048	Plastic 0 degrees	None

Result and analysis

Historical imagery reveals that slope failure starts from 2008 and failure is increasing with the span of time as shown in Figure 5. Visualizing the Google images of 2008 to 2021, it is understood that the size and volume of landslide has been significantly increased during the time period.

The major orientations of the joint sets of phyllites are listed as follows: Joint set 1: 376/26, Joint set 2: 244/56, Joint set 3: 195/68 (DD/DA). The mode of failures on the basis of kinematics analysis was tabulated in Table 2. The critical discontinuities with reference to natural hillslope, different mode of rock slope stability was calculated which was based on (Hoek and Bray, 1989) and (Goodman, 1976). The Rocscience software Dips 7 was used to perform kinematics analysis of hillslope.

There is possibility of wedge failure if the discontinuity intersection vector falls within the critical wedge region, which is bounded by the great circle representing the dip of the slope face and the circle representing the angle of internal friction (Φ). Figure 6 shows the wedges between J1 and J2 within the friction circle are vulnerable to wedge failure. Even, the kinematic analysis is useful tool for determination of planar, toppling and wedge failure, it doesnot consider the force acting on the plane and other parameters like height and geotechnical properties of materials like cohesion, unit weight and so on. This limitation was compensated by finite element method using rocscience software Phase2.

The maximum pricipal stress (σ_1) and minimum principal stress (σ_3) of the study area was simulated and are shown in Figure

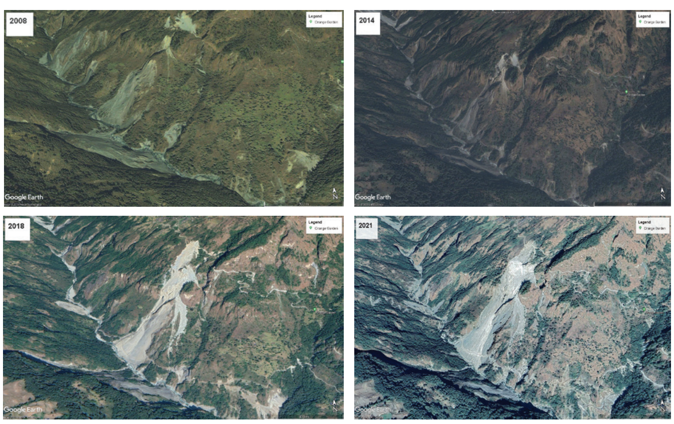

Fig. 5: Historical image of landslide from 2008 to 2021.

Table 2: Kinematics analysis of prominent Joint-sets and types of failures.

S.N.	Planar sliding	Wedge sliding	Direct toppling	Flexuaral toppling	Unstable slope
		Critical (%)			
1.	25%	66%	33.33%	8.33%	
2.	Foliation (dip direction/dip amount)				376/26
3.	Joint, J1 (dip direction/dip amount)				244/56
4.	Joint, J2 (dip direction/dip amount)				195/68
5.	Friction Angle: 28°				
6.	Lateral Angle: 30°				

7a,b respectively. As explained by Hoek and Brown (Hoek and Brown, 2019), maximum σ_1 and σ_3 plays imporatnat role rockmass failure. As per simulation, maximum σ_1 and σ_2 on the ground surface is 7.5 MPa and 3 MPa respectively. The relation between σ_1 and σ_3 is explained in Equation 1.

In finite element method of hillslope analysis, the strength factor is an important parameter used to represent the material properties of the rock and soil used to represent the material prperties of the soil or rock. The strength factor is typically defined as the ratio of the maximum shear stress that a rock or soil withstand without undergoing failure. In particular case, a low strength factor (<1) can lead to slope instability and failure, while a high shear strength (>1) can lead to less deformation and more stable slope. The strength strength factor is less than 1 is from 1500 amsl to top which signifies possibility of failure and high deformation (Fig. 8). The unstable section is nearly 15 m in depth which is due to fractured and jointed rockmass.

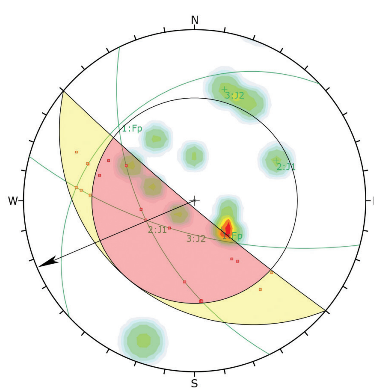

Fig. 6: Kinematic analysis of prominent joint sets.

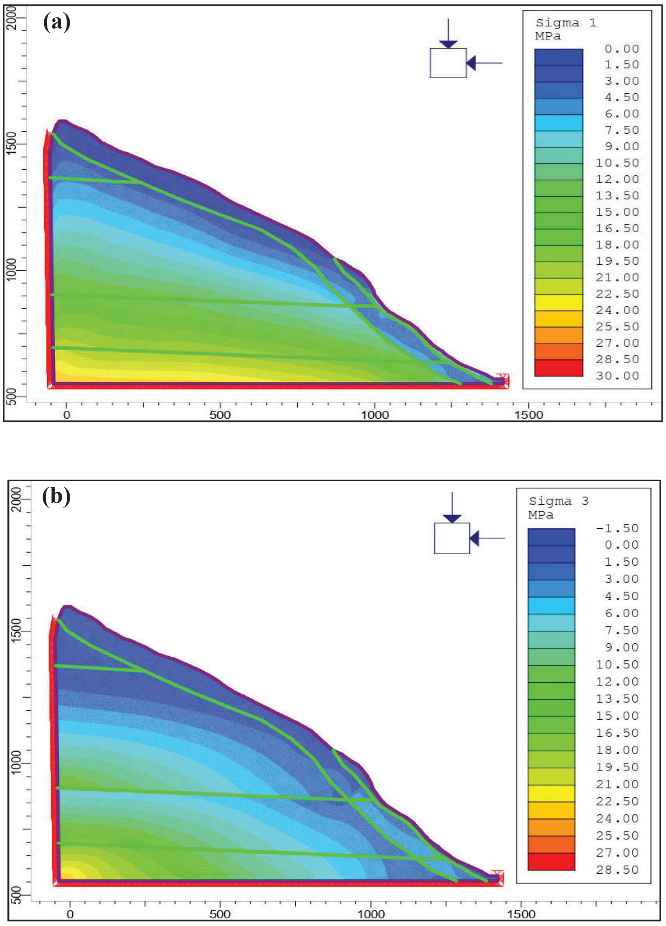

Fig. 7: Distribution of stresses along the profiles (a) σ_1, (b) σ_3.

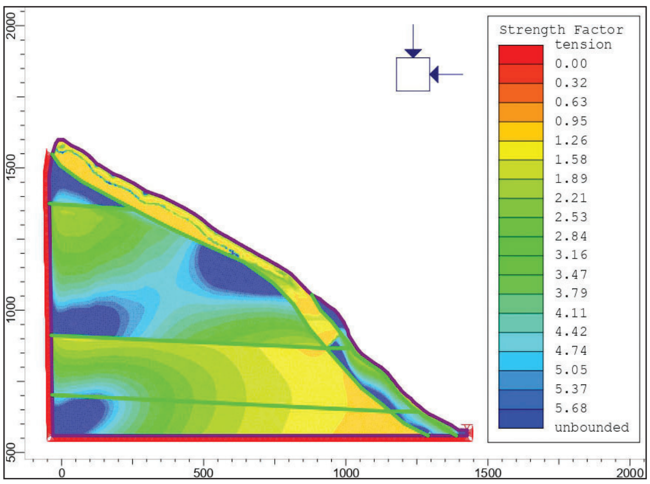

Fig. 8: Strength factor of material.

Table 3: Discontinuity data and failure conditions.

Hill-slope	Dip dir./ angle (°)	Failure conditions		
		Plane	Wedge	Toppling
220/86	376/26 (Fp)	No	Fp & J1, Fp & J2	-
	244/56 (J1)	Yes		Yes
	195/68 (J2)	Yes		Yes

***Note**: Fp = Failure plane*

DISCUSSION

On the basis of google earth and historical imagery, Bukula landslide was generated before 2008 and was progressive in the span of time. Geological mapping and ERT survey between 2013 and 2019 also revealed that the size of landslide was increased (NEA, 2015, 2019). From the map of DMG (2002), there is thrust between Dandagaon Phyllite and Kunchha Formation. The sheared zone of 25 cm and trend 285 and plunge 68 was observed on the upstream section of Bukula landslide (NEA, 2013). Therefore, the factors of rock slope failures are fragile geological condition, river toe cutting and cutslope for road construction. The modes of failure based on kinematics analysis are given in Table 3 and the probability for plane failure is 25%, wedge failure is 66%, and toppling failure is 33% with a maximum safe slope angle found as 55°.

The displacement based on numerical analysis of hillslope showed that the maximum deformation is 30.67 cm in 1600 amsl. The deformation gradually increases from 1200 to 1600 amsl and decreases in amount when the elevation increases. The strength factor is also minimum in 1600 amsl. Therefore, low strength factor and high deformation matches with the simulation are shown in figures 8 and 9. The kinematic analysis also showed the potential of failure. In addition, the google imagery also reveals the failure in the middle section of the hillslope.

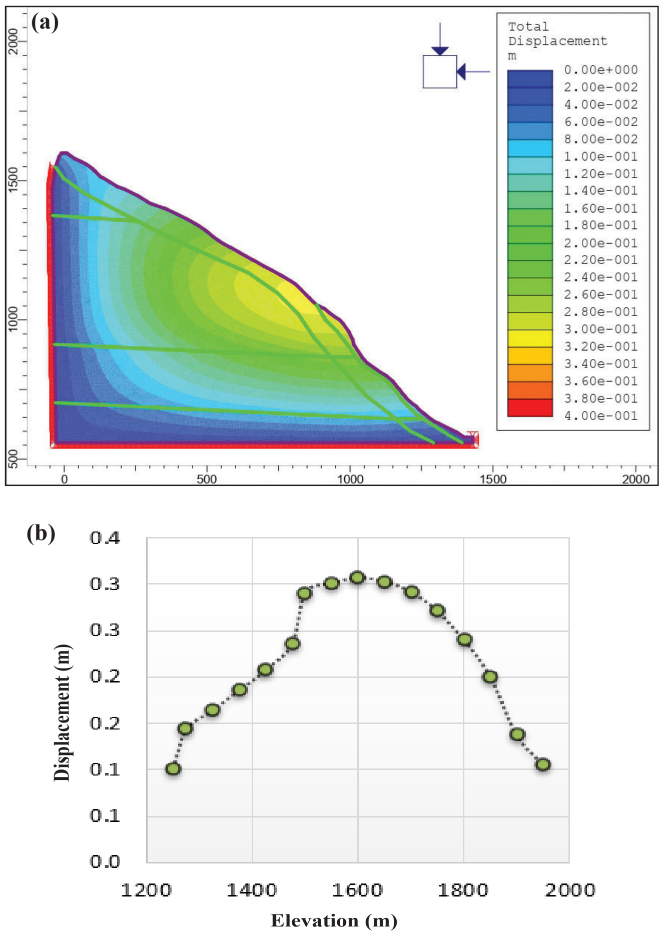

Fig. 9: Hill-slope condition (a) total displacement of materials, (b) elevation vs displacement.

CONCLUSION

It is concluded that the landslide is significantly increased from 2008 to present. The type of failure is mosty wedge failure with occassional plane failure. However, buckling type failure was also observed in site investigation in poor rock mass condition. The strength factor of middle section is below one and there is higher deformation and dispalcement. The total displacement at 1600 amsl is 30.67 cm. Maximum σ_1 and σ_3 on the ground surface is 7.5 MPa and 3 MPa respectively. Bukula landslide was triggered by sheared rockmass, river toe cutting and cut slope slope for road construction. Hence, borehole drilling and installation of peizometer was recommended for detail analysis of ground condition.The problem due to landslide should be mitigated by cutting the slope below 55°, installing proper support installment and designing proper drainage system.

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