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Progressive Reactivation of the Baliyanala Landslide: Geological Controls and 3D Numerical Assessment

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Abstract

The Baliyanala landslide is a long-lived, progressive landslide that was initially triggered in the year 1882 and has since undergone multiple episodes of deformation and reactivation. The landslide is developed within a Neoproterozoic metasedimentary Krol Formation, that has experienced significant tectonic activity. The geological framework is characterized by pervasive foliation, closely spaced discontinuities, and fault damage zone, which collectively impart pronounced mechanical anisotropy to the rock mass. Consequently, slope deformation at the Baliyanala landslide is spatially heterogeneous, with displacement patterns governed by the orientation, persistence, and interaction of anisotropic fabrics. Fault related damage has further reduced shear strength of the rock mass subjected to groundwater interaction, facilitating progressive weakening, localized strain accumulation, and repeated reactivation under varying stress conditions. Slope stability analysis was carried out using three-dimensional finite element modeling (3D FEM) in RS3 software, where anisotropic rock mass behaviour was explicitly represented through weak plane and ubiquitous-joint formulations. The numerical models successfully captured stress–strain localization, progressive failure mechanisms, and the spatial variability of deformation observed in the field. Model results enabled the identification of both actively deforming zones and areas susceptible to future instability and were found to be in good agreement with documented deformation patterns within the study area. This integrated geological and numerical approach provides valuable insights into the long-term evolution of progressive landslides in fault-damaged metasedimentary terrains and supports improved assessment of slope instability hazards.

Keywords

Landslide, Slope stability analysis, Finite Element Modelling, Fault Damage Zone.

1 Introduction

Slope stability analysis is a cornerstone of geological and geotechnical engineering because several slope instabilities pose serious threats to human life, infrastructure, and the environment (Hungr et al. 2014; Cruden & Varnes, 1996). Accurate prediction of the behaviour of the slope is the key factor for designing safe infrastructures, for planning mitigation strategies and constructing various retaining structures in the mountainous regions like Himalaya. Traditional 2D limit equilibrium methods have been utilised for their simplicity and computational efficiency. However, these approaches depend on the presumption of plane strain conditions and uniform slope geometry, which rarely reflect the complexities in the typical landslide. (Griffiths & Marquez, 2007). As a result, two-dimensional analyses frequently simplify the geometry of the slope, neglect the influence of three-dimensional joint networks, and generally estimates conservative factor of safety. (Zheng et al. 2015; Duncan & wright 2005). This may result into to either overdesigning or misinterpretation of potential failure mechanisms. Conversely, three-dimensional slope stability numerical methods viz. Finite Element Method (FEM)based RS3 software has the provision of incorporation of realistic slope geometries, different material properties and varying joint orientations (Rocscience, 2020; Chen et al. 2025; Cheng et al. 2025). Using the ubiquitous joint approach in a 3D framework enablesto model the anisotropic behaviour of jointed rock masses and identify potential vulnerable failure areas more precisely that remain concealed in two-dimensional slope stability evaluations (Jing, 2003). Recent findings indicates that 3D slope stability analysis frequently gives superior and more precise factor of safety compared to 2D approaches, while providing valuable insights focusing on patterns of displacement and causes of failures. (Li et al. 2019). The transition from conventional 2D slope stability techniques to advanced 3D modelling represents a substantial improvement in enhancing the precision of landslide hazard evaluations and the design of mitigation strategies. In this study, the use of the ubiquitous joint approach to significant landslide problems is demonstrated in comparison to the regular three-dimensional stability analysis, as is the effectiveness of this technique in solving such challenges within a three-dimensional framework.

1.1 Study area

The Baliyanala landslide (Fig. 1) in Nainital constitutes a significant and intricate slope failure system that has demonstrated both progressive and retrogressive instability modes since its onset in 1880. The landslide, marked by multidirectional failure tendencies, has experienced repeated movements despite numerous stabilisation attempts, highlighting its persistent and recurring nature. The landslide, measuring approximately 400 m in height and 450 m in width, qualifies as a large-scale mass movement due to its volume and areal extent. The extensive history of intermittent failures underscores the necessity of comprehending the fundamental geological and geotechnical factors, as traditional stabilisation methods have proven inadequate in halting its activity. The magnitude, configuration, and frequency of failure incidents render the Baliyanala landslide a pivotal case for sophisticated slope stability evaluation.

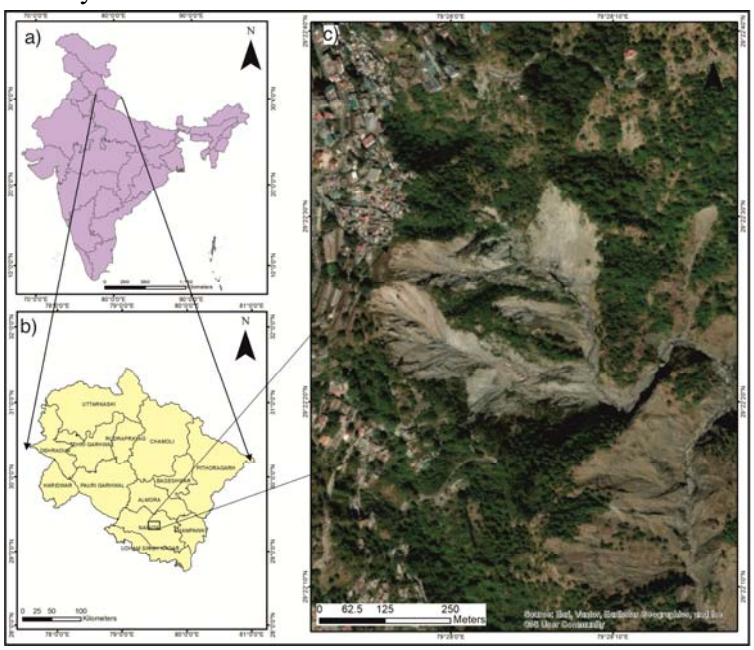


Fig. 1 Location map of the study area a) State map of India b) Uttarakhand district map highlighting study area c) Satellite image showing Baliyanala landslide.

1.2 Geology of the study area

The region under consideration falls within the Outer Lesser Himalaya, bordered by the Sher-Ka-Danda Ridge to the northeast, the Ayarpatta Ridge to the southwest, and the China Peaks to the northwest. The Nainital city is situated north of the Main Boundary Thrust (MBT) and it is intersected by the Nainital Lake fault running from the centre of Nainital Lake. Besides the aforementioned structures, multiple branches of the NLF are also observed in this area. The creation of Nainital Lake is attributed to the rotational displacement along the lake fault, estimated to have occurred around40-50 Ka. Krol Formation consists of Dolomitic limestone, rhythmite, silty sandstone and purple slates. The region contains the following lithological units: Krol-A is composed of Manora slate, Krol-B is composed of Hanumangarhi slate, Krol-C is composed of dolomitic limestone and silty sandstone, Krol-D is composed of thickly stratified dolomitic limestone, and Krol-E is composed of rhythmite and dolomitic limestone (Kumar et al. 2017).

2 Methodology

In order to conduct a 3D slope stability investigation, this study implemented an approach that incorporates finite element-based 3D numerical modeling, Unmanned Aerial Vehicle (UAV)terrain mapping, point cloud data processing, field investigation and subsurface characterization using borehole logs and Seismic Refraction Tomography (SRT). The slope geometry was obtained from the point cloud data was used to construct a geometry of the slope. Subsequently, the material properties derived from laboratory investigation, geological and geotechnical characterization of the material in the field were attributed to the corresponding lithological layers in terms of shear strength parameters, including unit weight, cohesion, angle of internal friction, young's modulus, and poisson's ratio. In order to assess potential failure mechanisms and critical zones of instability, the three-dimensional slope stability analysis was conducted in RS3 considering various scenarios. This multi-dimensional approach to slope stability studies improves the reliability of stability assessments and generates a comprehensive output that facilitates the better understanding of the slope instability.

2.1 Field Investigation

Field investigation was conducted in two stages. The first stage involved terrain mapping of the entire study area and the adjoining area while the second stage focused on detailed geological and geotechnical investigations.

2.1.1 Terrain mapping and point cloud processing

High-resolution aerial imageries were acquired using a DJI Phantom 4 UAV, which has demonstrated its efficiency in landslide mapping by generating dense point cloud datasets by photogrammetry technique (Singh et al. 2025). The raw point cloud data was improved and examined using CloudCompare software. This included georeferencing, reduction of noise, smoothening and segmentation of the data. As a result, spatial accuracy was ensured and the unique features and area of the landslide was highlighted (CloudCompare, 2023). For reconstruction of the surface, removal of excess points and simplification of the data MeshLab software was used. A realistic 3D mesh was generated (Fig. 2) which reduced computational challenges while maintaining the geomorphological features of the landslide (Cignoni et al. 2008).

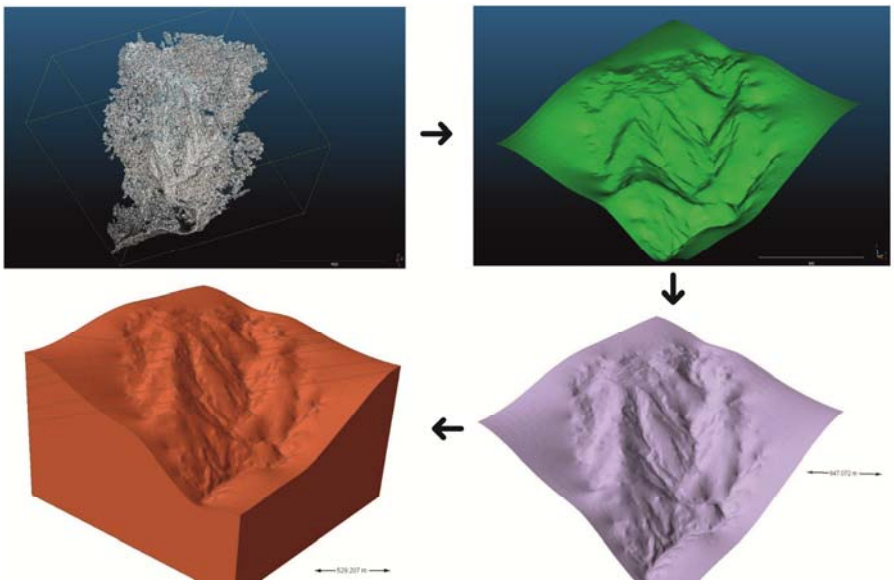


Fig. 2 Workflow of generation of 3D slope model for stability analysis.

2.1.2 Geological and Geotechnical Investigations

Detailed geological and geotechnical investigations were conducted in the study area to understand and characterize the materials contributing to the instability, structural features present, and evaluate the quality of the rock mass. This landslide exemplifies episodic complex failure event involving the collapse of both debris material and bedrock during each failure. Six distinct material units were identified and mapped in the field during the field visit, Comprises of overburden debris, highly weathered dolomitic limestone, dolomitic limestone, rhythmite, greenish slate and greyish splintery slates. Highly weathered Dolomitic limestone overlained by debris material was observed in the crown region, underlain by a rhythmite sequence approximately 25–30 m thick. The slate formations were observed in the middle and toe regions of the landslide. Bedding and foliation planes exhibited dip angles ranging between 20° and 35°. Comprehensive rock mass classification viz. Rock Mass Rating (RMR) and Geological Strength Index (GSI) was determined by collecting data from the exposed bedrock. Orientations of the discontinuities were documented using Brunton compass (Table 1), additionally spacing, aperture, persistence surface conditions and joint roughness coefficient (JRC) of the discontinuities were systematically documented. To estimate intact rock strength, Schmidt rebound hammer tests were conducted. Kinematic analysis was carried out to evaluate potential failure modes in relation to discontinuity orientations. RMR and GSI were obtained by analysing the above-mentioneddiscontinuity conditions, surface roughness, and weathering characteristics of the rockmass.

Table 1. Orientation of discontinuities present in the slope.

Material	Joint set	Dip amount (°)	Dip direction (°)
Dolomitic limestone	J1	35	290
	J2	50	50
	J3	65	180
Greenish slate	J1	20	260
	J2	83	100
	J3	85	40
Grey slate	J1	15	240
	J2	25	355
	J3	50	185

2.2 Slope stability analysis

The presence of jointed rock mass exhibits very complex behaviour because of the adverse orientation of the discontinuities present in the rock mass and it substantially influence the strength and deformation of the slope mass. In comparison with the intact rocks these rock mass units are highly anisotropic and heterogenous in terms of shear strength and stiffness (Hoek & Brown, 1997). Generally, these discontinuities act as a failure surfaces, when they come in interaction with the groundwater, under seismic loading and intense weathering and it significantly triggers the instability in the slopes (Carranza-Torres & Fairhurst, 2000). For large scale instability problems, it is very difficult and challenging to determine the shear strength parameters through traditional laboratory experiments can be time consuming hence, requires more efficient field-based approaches. In this

study, the uniaxial compressive strength (UCS) was estimated using indirect method by performing in situ Schmidt rebound hammer test which is a rapid and very reliable method to assess the intact rock strength (Aydin & Basu, 2005). The Geological Strength Index (GSI) was subsequently evaluated using the Hoek and Bray chart, enabling classification of rock mass quality based on structural features and surface conditions (Hoek & Brown, 1997; Hoek et al. 1998).

These parameters were then converted into equivalent Mohr–Coulomb shear strength parameters viz. cohesion and angle of internal friction angle through RSData software, which applies the Hoek–Brown criterion to derive parameters suitable for numerical modeling (Hoek et al. 2002; Huang et al. 2002). The derived values were incorporated into three-dimensional slope stability analyses in RS3 using the ubiquitous joint method, which embeds joint behaviour within a continuum framework to represent anisotropy without explicit block discretization (Rocscience, 2020). Geological field data were integrated, and analyses were performed under static, saturated and seismic saturated conditions.

Table 2. Parameters used in stability analysis.

Material	**Cohesion (kPa)**	**Angle of internal friction (°)**	**Ru**
Debris	2	27	0.2
Weathered dolomitic limestone	400	22	0.1
Dolomitic Limestone	606	23	0.1
Rhythmite	165	21	0.1
Greenish slate	216	22	0.1
Grey slate	300	25	0.1

After constructing the three-dimensional slope model in RS3, the geological layers were carefully defined by introducing cut planes to delineate the different lithological units within the slope geometry (Fig. 3).The layers were divided based on the lithological boundaries identified in the field, SRT and Borehole log data which ensures the actual representation of surface morphology and subsurface heterogeneity (Palmer, 1981; Jongmans & Garambois, 2007).

Each material zone was then assigned its respective shear strength parameters, derived from field investigations and subsequent conversion into equivalent Mohr–Coulomb values. These parameters, summarized in (Table 2), ensured that the heterogeneity of the slope was realistically represented in the numerical framework. To evaluate the influence of discontinuities on the outputs of stability analysis, two cases of simulations were conducted: one incorporating the ubiquitous joint method to explicitly account for anisotropic joint behaviour, and another without ubiquitous joints to represent a continuum-only condition. Within each modeling approach, four distinct scenarios were analyzed to assess slope stability in the absence of protective measures. These scenarios included static loading, static saturated conditions (through pore pressure ratio R_u), and seismic conditions.

The boundary conditions during the modelled were restrained using auto surface-restrained option which enables the bottom and sides of the external boundary is assigned XYZ restraints, and the top surface (ground surface) has no restraints. The modelled was meshed with a fine four-noded tetrahedral mesh which divides entire model into approximately 25000 elements, allowing for accurate simulation of stress redistribution, displacement patterns, and potential failure surfaces. This integrated approach provides a more simplistic and reliable alternative to laboratory testing, producing accurate slope stability assessments in complicated landslide situations.

This systematic modeling strategy provided a comparative framework to quantify the role of joint networks in slope instability and highlighted the necessity of incorporating joint mechanics for reliable hazard assessment in complex landslide environments.

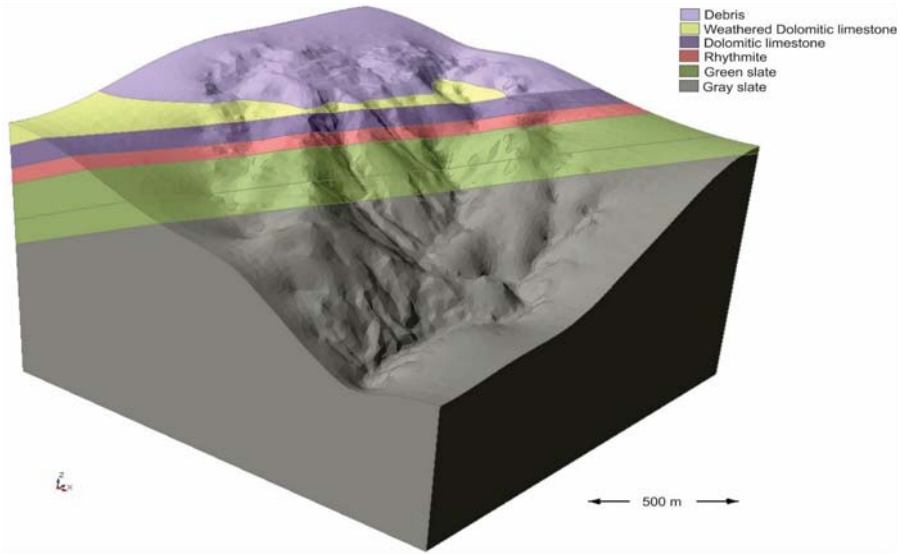


Fig. 3 Final 3D slope geometry depicting different lithology layers.

3 Result and discussions

The slope stability assessment was done in RS3, offered insightful information on the behaviour of the slope under different scenarios. Two cases of stability analysis were performed in this study, 1. Considering rockmass as a continuum with isotropic shear strength properties i.e. without taking joints/discontinuities into account and 2. Considering the ubiquitous joint network in the stability assessment. The outcomes of both numerical simulations are presented and discussed below.

3.1 Case 1

The results of case 1 numerical simulations indicate relatively higher factor of safety and uniform displacement patterns (Fig. 4). The absence of joint networks limited the ability of the numerical model to capture the anisotropic behaviour of the material. Similarly, the failure surface predicted in this case of simulation are smooth and less complex often confined to shallow translational modes. While this approach provided a simplified understanding of slope performance, it underestimated the potential for localized block movement and progressive failure along discontinuities.

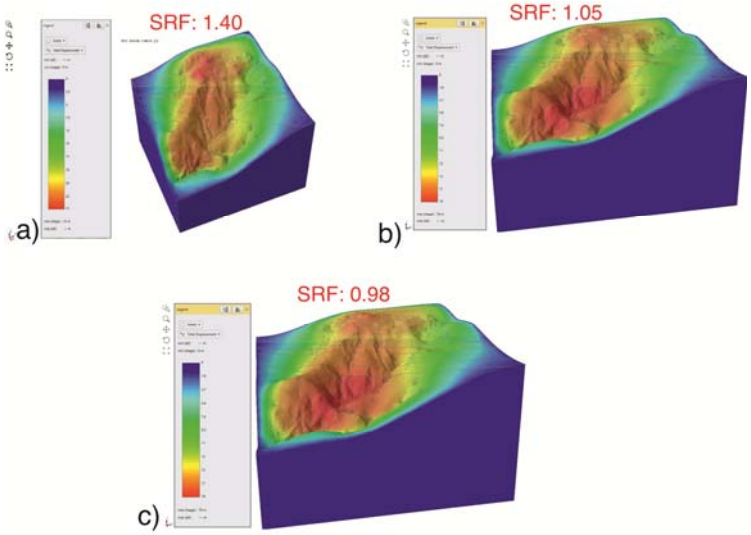


Fig. 4 Results of stability analysis without ubiquotous joints showing total displacement under different scenarios a) dry, b) saturated c) dynamic.

3.2 Case 2

On the other hand, the second case of numerical analyses consisting of joint network were performed in a continuum framework. This approach allows the model top realistically represent anisotropy in the slope mass by accounting joint parameters. The results obtained from these analyses shows comparatively lower factor of safety to the first case and exhibits multidirectional tendencies of the

failure which is quiet the case in the baliyanala landslide (Fig. 5). The field observations also corroborate the results obtained by incorporating the joint network. Displacement vectors revealed localized block rotations and separations, consistent with field observations of episodic failures in the study area. The influence of groundwater and seismic loading was particularly pronounced in the ubiquitous joint simulations, where pore pressure and dynamic forces accelerated shear displacement along pre-existing planes of weakness. The outcomes of this case highlight the critical role of discontinuity and their anisotropy in the stability analysis and also and highlight the limitations of traditional continuum-based studies.

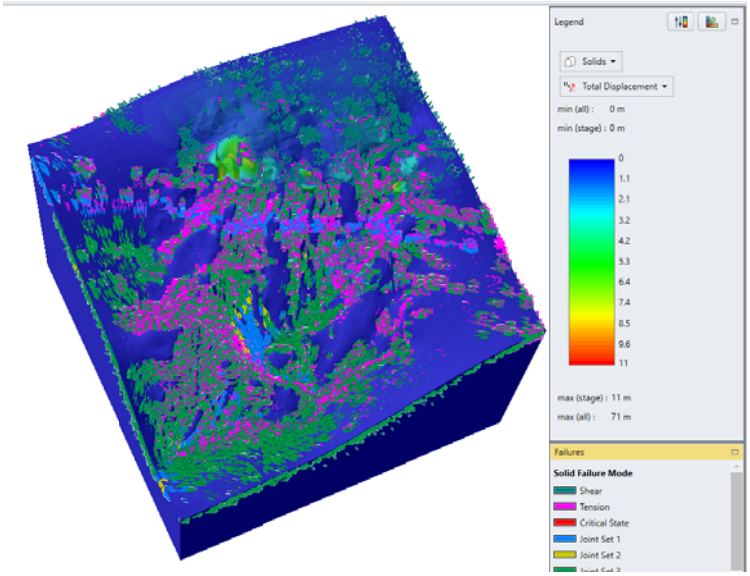


Fig. 5 Slope stability analysis results of case 2 in saturated condition showing structurally controlled failures in the slope.

Overall, the comparison between both the cases shows that the model without ubiquitous joints may offer conservative estimates of factor of safety (Table 3) of the slopes and may fail to represent the realistic failure mechanism in a complex slope framework while the other case depicts the realistic behaviour of the slope masses particularly under saturated and dynamic conditions where discontinuity-controlled failure predominates.

Table 3. Factor of safety obtained from slope stability analysis under different scenarios

Scenario	**Case 1**	**Case 2**
Static	1.40	1.14
Saturated (Ru)	1.05	1.01
Dynamic	0.98	0.91

4 Conclusion

The integration of joint network in a 3D slope stability analysis significantly improves the reliability of slope stability analysis and reflection of true instability mechanism. The homogenous models are simpler but generally overestimates the factor of safety and neglects the influence of anisotropy of the discontinuities. In contrast, joint simulations capture the complex failure mechanisms observed in the field, including progressive and multidirectional failures, block rotations, and displacement along persistent joint planes. The results confirm that discontinuity-controlled behaviour is the dominant factor in large landslides such as Baliyanala, and that advanced numerical approaches are essential for realistic hazard assessment and the design of effective mitigation measures.

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